A FRAMEWORK FOR THE OPERATIONAL WATER QUANTITY MODEL

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INTRODUCTION:

As a part of the D.O.A. project coupled with the in-house Kissimmee project (Program No. 8430), this particular study is oriented toward providing a tool to examine the operational behavior of the Kissimmee water system with main emphasis on the development of an overall framework of the water quantity model. More specifically, the objectives of this study are:

a. To generate 3-hour discharge data for the ten year period (January 1961 to December 1970) for 19 planning units of the Kissimmee Basin using the available basin parameters, stage conditions and daily rainfall inputs in the FCD sub-basin model;

b. To verify generated data in all possible ways;

c. To develop routing methodologies to distribute these generated values through the Kissimmee Basin system of lakes, canals, and control structures;

d. To obtain stages and gate opening data coupled with initial conditions at all control structures;

e. To formulate the backwater functions for all the channel reaches of the main stem Kissimmee system;

f. To compile and to develop (if necessary) the formulation of lake stage-storage relationships, control structure rating curves, and channel ratings for the upper and lower Kissimmee basin;

g. To design a generalized computer program to include all of the routing steps;

h. To evaluate the results of the routing model, and

i. To cite areas for further investigation (if any).

NEED OF OPERATIONAL WATERSHED MODELS

Although all the conventional models are developed with different purposes, methodologies, tools and settings, they are usually conceptually valid for natural hydrologic drainage systems. Therefore, it seems that these models have to be modified in some fashion to analyze controlled water systems of the FCD in general and the lake, channel and control structure systems of the Kissimmee basin, in particular. It appears that this particular, practical thinking created a need for the development of an operational watershed model (also known as water quantity model) to include operational characteristics of the FCD water system coupled with theoretical and experimental data for formulating basic hydrologic processes.

From the analytical framework standpoint, simulation and optmization techniques with stochastic and deterministic inputs are being used in planning and design of water systems. Considering the necessary assumptions and speculated conditions required for reaching a mathematical solution in most cases, these design models give general answers to the overall problem and do not generate the most desired product for operational needs (10). As pointed out by Lindahl and Hamrick, operationally oriented models must give specific answers to very specific questions and circumstances. It becomes essential to develop a practical model (with adequate theoretical basis) to function as a short term and long term decision-making aid within an operational framework and within existing periopheral monitoring capabilities for the typical water system of the FCD. Accordingly, a program was initiated to develop models in this direction (9, 10). The description of the structure, component parts, and past and present developmental procedures associated with these models, is briefly attempted in the following sections.

COMPONENTS OF THE FCD WATER QUANTITY MODEL

From an operational standpoint, the basic objectives of the FCD Water Quantity Model (often called the operational watershed model) are:

a. To generate the discharge-time curves at various points in the system,

b. To simulate stages at both sides of various control points or structures (16, 17), and

c. To include operational parameters (such as set of gate operations) in the overall simulation methodology to generate pnactical information sought in a and b.

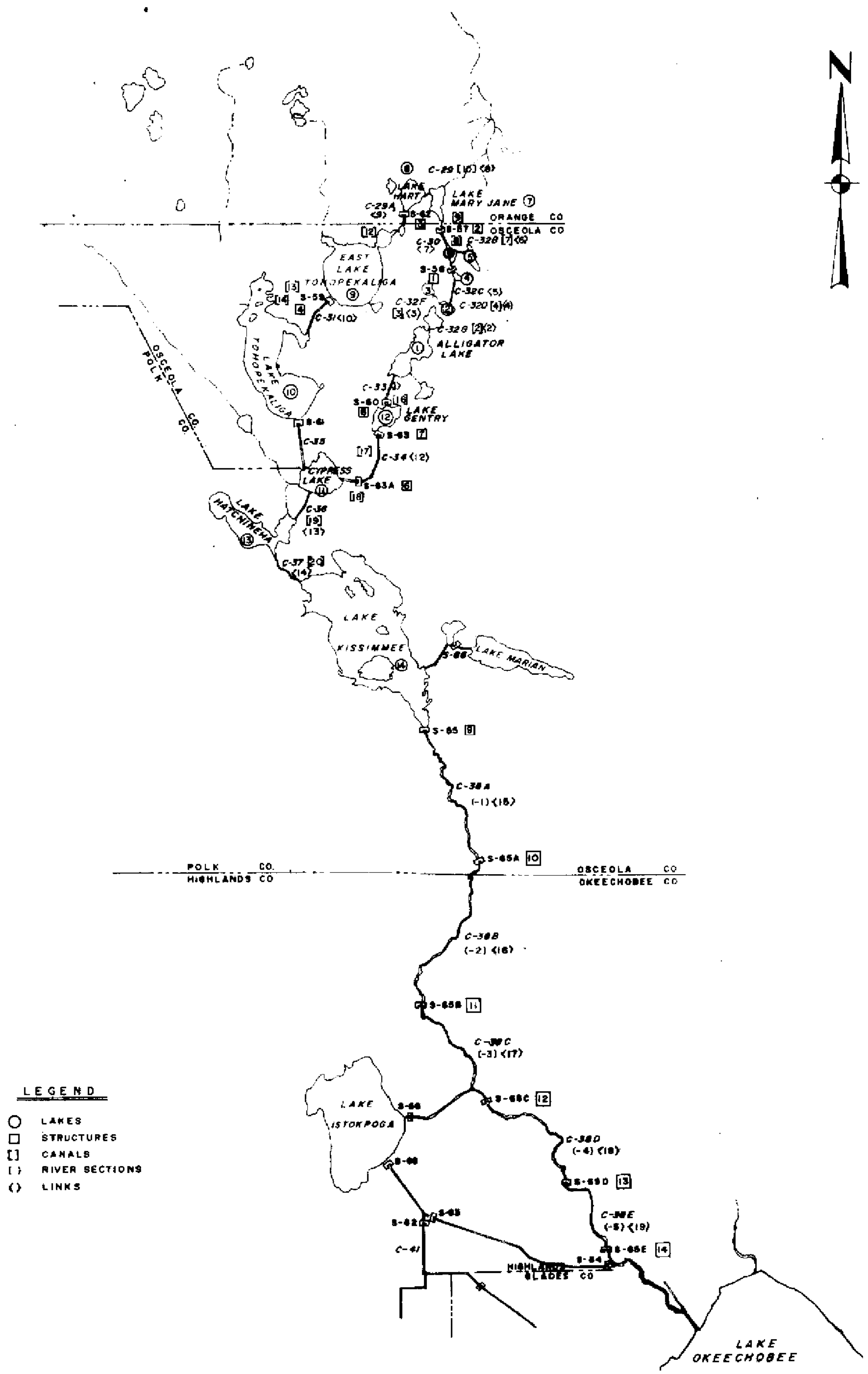
To develop a methodology in these directions, a controlled and typical water system (with lakes, channels, channelized river and control structures) of the Kissimmee basin is first selected. This whole Kissimmee Basin area of 3,000 square miles is divided into 19 sub-basin units based on the drainage characteristics of these areas. The size and nature of the Kissimmee River basin with its structural components and its subdivisions into planning units are depicted in Figures 1 and 2. Having broken down the Kissimmee basin into 19 drainage areas, the next obvious questions in light of our final objectives are

a. What are the key hydrologic components of the terrestrial branch of the hydrologic cycle for each of these planning units?

b. How much water is contributed to various hydrologic processes from the rainfall inputs derived from the existing rain gaging network?

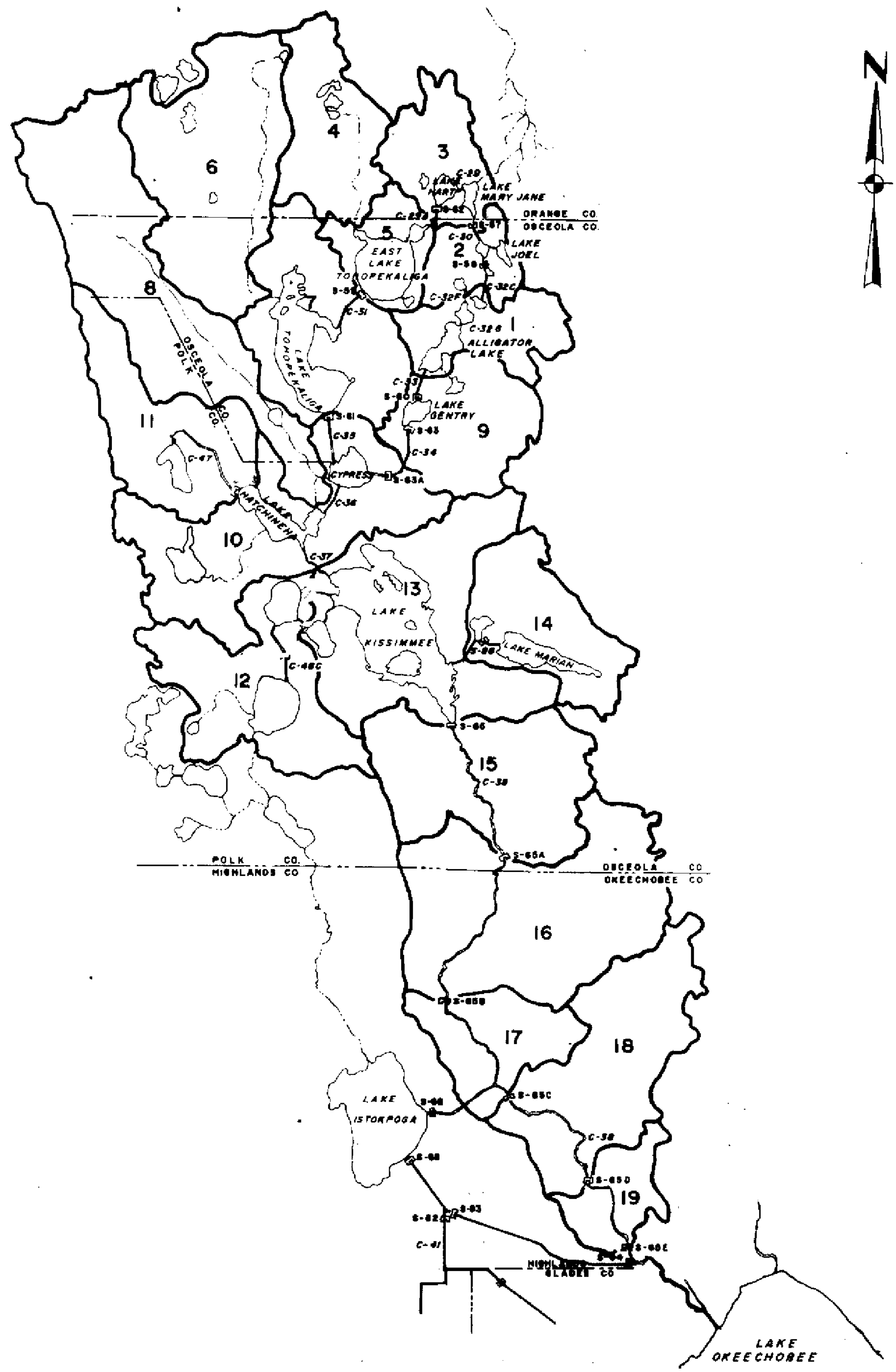
c. How much water flows out from each of these planning units, and

d. In what way does the water in different processes get distributed through the controlled system of lakes, channels and operating gates?



CHAIN OF UPPER KISSIMMEE LAKES AND LOWER KISSIMMEE FIVE POOLS.

FIGURE 1



KISSIMMEE RIVER BASIN AND THE FCD SUB-BASINS.

FIGURE 2

To provide some answers to these basic questions, an attempt is made to develop the FCD Water Quantity Model in three stages and thus, its developmental procedure is broken down into the following three component parts:

1. Sub-basin model,
2. Routing procedure, and
3. A routing methodology to couple the routing technique with the sub-basin model.

Basic computational steps involved in our FCD Operational Watershed Model (including the above-mentioned three component parts) are outlined in Figure 3.

DESCRIPTION OF THE SUB-BASIN MODEL

The basic foundation on which the FCD sub-basin model was developed and modified is essentially a parametric approach for formulating the physical system of the Kissimmee basin in terms of hydraulic simulation (1, 7, 9, 10, 16, 17, 18). Considering the land phase of a hydrologic cycle, a conceptual watershed model (also known as a simplified catchment model) is first outlined by identifying various realistic hydrologic processes applicable to the Kissimmee sub-basins under investigation. This flow diagram is shown in Figure 4. As depicted in Figure 4, the rainfall event becomes a main driving force for triggering the component parts (such as surface storage, overland flow, channel flow, flow through soil reservoirs, water losses and basin outflow) of the Kissimmee sub-basins. To evaluate each of the processes in a quantitative manner, a classical parametric approach is used to evaluate spatial and time distribution of inputs in various hydrologic processes. In this approach, among various available formulations for estimating the water quantities associated with these surface and sub-surface components, empirical relationships with

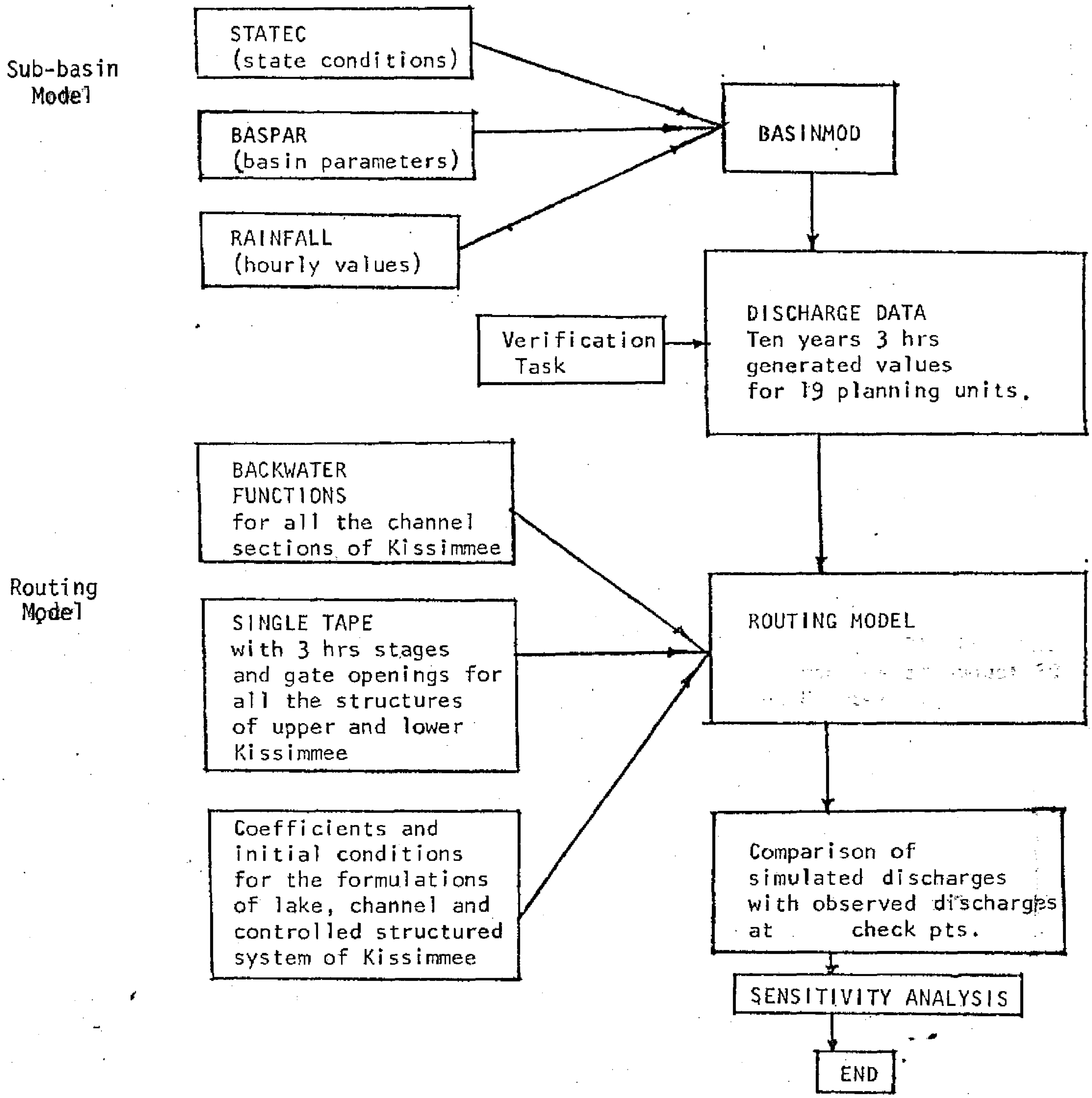


Figure 3. Flow chart of major computational steps involved in the FCD water quantity model

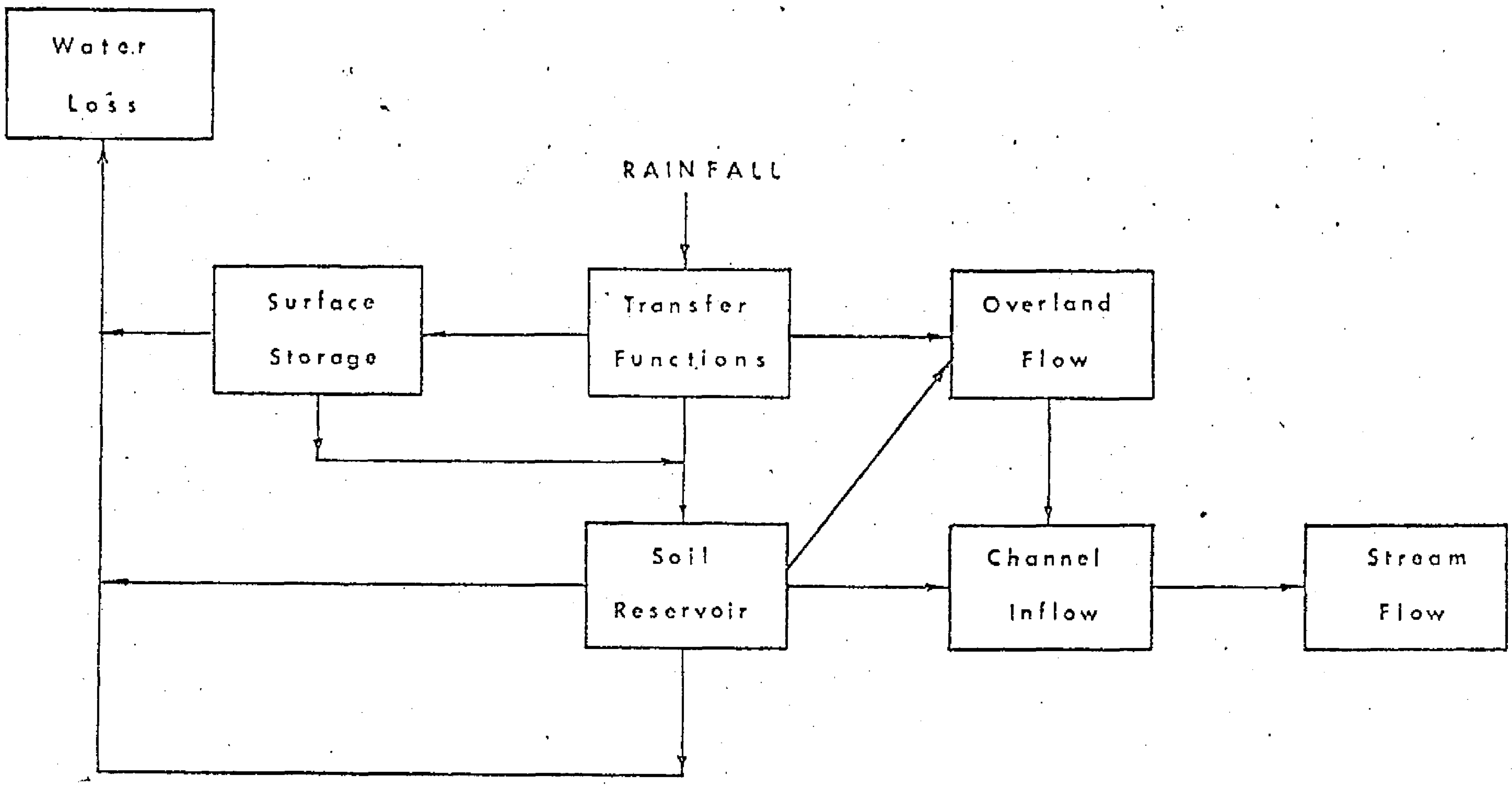


FIGURE 4. A Simplified Conceptual Watershed Model

parameters reflecting the physical characteristics of soil, vegetation types and retention properties of the basin are selected. These empirical relationships are largely based on field and laboratory experiments coupled with climatological, hydrological and topographical observations (4, 5, 6, 7, 16, 17, 18, 19).

COMPUTATIONAL STEPS OF THE SUB-BASIN MODEL

For computational clarity, the response phases to rainfall inputs depicted in Figure 4 are rearranged in more detail as shown in Figure 4A. It can be seen from Figure 4A that the major computational steps are related to:

a. Processing of input rainfall values,

b. Formulations of infiltration phenomenon,

c. Surface storage and overland flow equations,

d. Estimation of water losses, and

e. Quantification and routing of sub-surface flow through a multi-layer soil system.

Since the detailed descriptions and discussions of rationale behind these formulations are given by Lindahl, Sinha, Hamrick and Khanal (7, 9, 10, 16, 17, 18), these formulations are briefly discussed in the following section of this summary report.

PROCESSING OF INPUT RAINFALL VALUES

Using the available network of raingaging stations over the entire Kissimmee River basin, daily rainfall values are obtained for each of the 19 planning units from the daily rainfall values of surrounding representative raingaging stations.

These recorded daily rainfall values are further synthesized to generate hourly values using linear stochastic model for the consecutive hourly rainfall record.

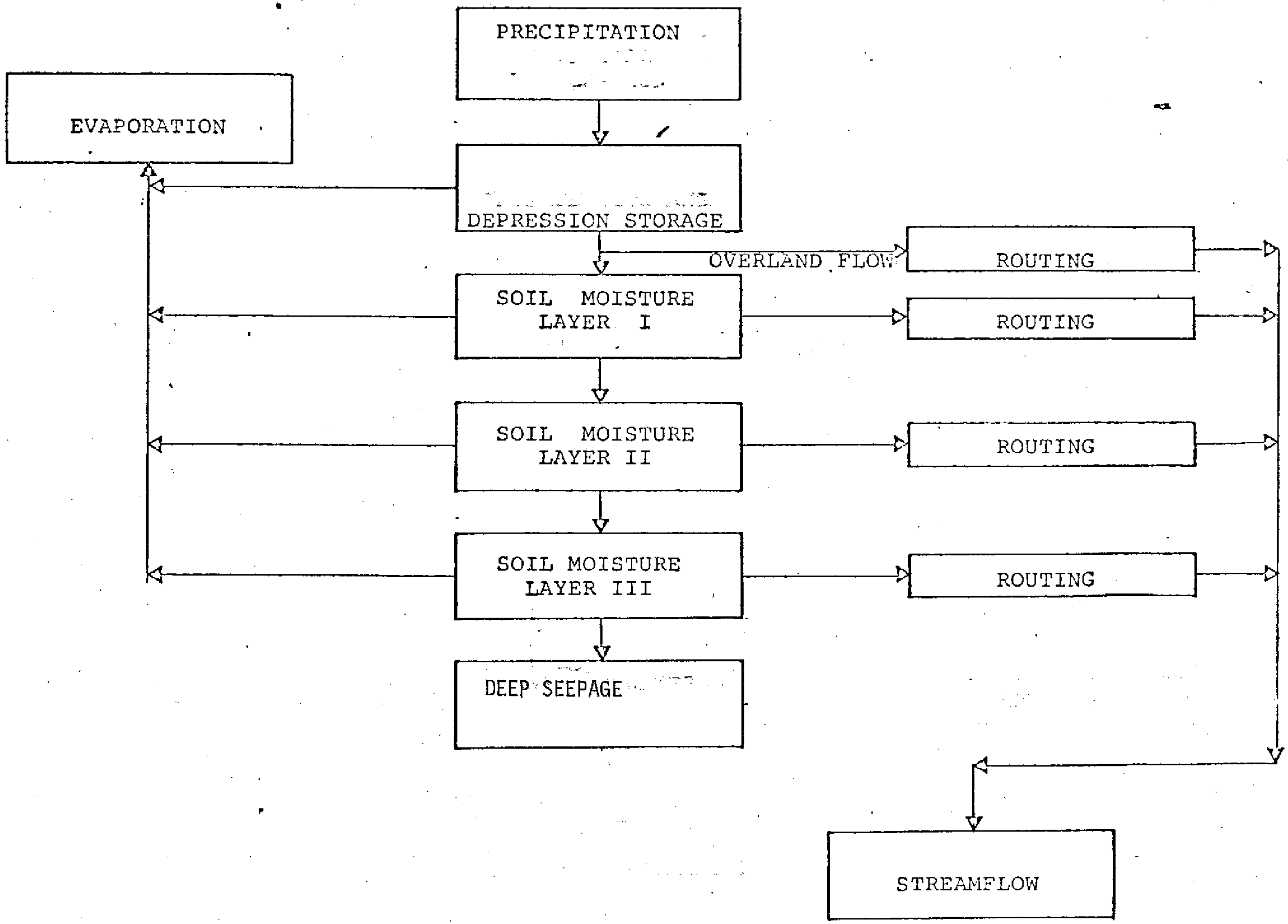


FIGURE 4A F. C. D.'S SUB-BASIN MODEL

FORMULATIONS FOR INFILTRATION PHENOMENON

Among various formulations and concepts proposed by many soil scientists, a modified form of the empirical equations originally developed by Holtan is used in quantifying the infiltration phenomenon. Such equations are:

$$f = A(SA)^{1.4} \quad \text{for } SA \geq G$$

and

$$f = A(SA)^{1.4} + FC \quad \text{for } SA < G$$

where

f = capacity rate of filtration,

A = surface penetration index,

SA = storage currently available in the soil reservoir,

FC = constant rate of infiltration between consecutive layers

G = total amount of free or gravitational water in a soil profile of selected depth (4, 5, 16, 17).

These equations are chosen for the following reasons:

1. availability of field data to estimate the coefficients of these equations,
2. adequate mathematical and theoretical basis, and
3. practical verification of these formulations on small experimental plots with various vegetation types (4,5).

SURFACE STORAGE AND OVERLAND FLOW

Besides infiltration, a part of precipitation is contributed to the storage in surface depressions. Such surface storage is computed as

$$VD = P - f \cdot DT$$

$$VDM = \text{maximum volume of surface storage} = d \sum_{i=1}^{N} bi$$

where

N = number of surface depressions,

DT = selected time interval

bi = area of ith surface depression, and

d = average depth of N surface depression (6,17)

After a part of precipitation input percolated into the ground and after a part filled the maximum volume of surface depressions, precipitation excess is contributed to overland flow. Mathematically, it is computed from simple subtraction as

Overland flow = OF = P - f when VD = VDM and P > f

where

P = precipitation input,

f = infiltration rate,

VD = amount of water currently stored in surface depressions,

VDM = maximum volume of surface storage (16, 17)

ESTIMATION OF WATER LOSSES

In the sub-basin model, water losses are considered as the part of precipitation input that reaches the ground surface but never appears at the watershed outlet (9, 16, 17, 18). With this definition, water loss can occur in different categories; i.e., water loss due to direct soil evaporation, evapotranspiration by existing vegetation and water loss due to deep percolation. These losses are in turn functions of various factors as shown in the following formulations:

1. Water loss due to indirect soil evaporation,

   $$\text{Loss 1} = C_1\left(1 - \frac{DWT}{DWTM}\right)\left[\frac{EP(NW)}{24}\right](DT)$$

2. Portion of water that is lost due primarily to the existing vegetation

   $$\text{Loss 2} = C_2(G_1)\left[\frac{EP\ (NW)}{24}\right] DT$$

3. Water loss due to deep percolation,

   $$\text{Loss 3} = (FC)\ (DT)$$

where

C_1 = ratio of maximum evapotranspiration to maximum pan evaporation value,

DWT = water table depth

$= \frac{(SA)\ (D)}{G}$

D = total depth of soil profile,

G = total amount of free gravity water that could exist in a soil profile,

DWTM = maximum depth to water table at which DWT will have a negligible contribution toward Loss 1,

EP = pan evaporation,

NW = number of weeks,

DT = time increment,

C_2 = constant = 0.78

G_1 = an overall growth index for the existing vegetation,

FC = constant rate of infiltration between consecutive layers

SA = storage currently available in reservoir

Adding these three losses together gives the total loss of water from a given soil profile. This value of total water loss is accounted for in estimating the recovery of water from the soil reservoir to the main channel.

QUANTIFICATION AND ROUTING OF SUB-SURFACE FLOW

The basic purpose of this computational step is to estimate the spatial and time connection of sub-surface flow from different soil reservoirs to the main channel. Thus, the first task is to determine the number of reservoirs. This is done by the reverse integration of the runoff hydrograph by establishing storage-flow relationships for a simple recession curve. Using this technique it is established that for our 19 planning units, soil profile can be represented by not more than three soil reservoirs. After determining the number of soil reservoirs, the basiccontinuity equation and a storage outflow curve is combined to provide contributions of each soil reservoir to the stream channel and also the total storage available in these reservoirs at the end of each time step. These computations take into account

1. the volume of water that is infiltrated during time DT,
2. initial available storage in a soil reservoir,
3. sum of water losses
4. volume of subsurface drainable water
5. time interval for the volume of the subsurface drainable water, and
6. the updated available storage

At the end of these computations, the discharges contributed by each soil layer and overland flow are obtained for each time interval. In the next step, these discharges are multiplied by the routing coefficients (which are estimated from Nash's routing equation) and resulting values are added together to obtain time distribution of streamflows at the watershed outlet.

INPUT DATA REQUIREMENTS

To carry out these computational steps for the 19 planning units of the Kissimmee basin, the parameters of the formulations should be known. Since these parameters represent the agricultural-related water characteristics of the basin, they are estimated based on the available research publications of the ARS and many reports delineating the regional characteristics (7, 9, 10, 16, 17, 18).

To compute infiltration characteristics, the appropriate basin parameters are:

a. Total available storage in three soil reservoirs (i.e., TAS(1), TAS(2) and TAS(3))

b. Constant rates of infiltration in three layered soil systems from one layer to another (designated as F(1), F(2), and F(3).

c. Total amount of gravitational water in these three layers (i.e., G(1), G(2), and G(3)).

d. Portion of G that can be drawn into surface water (i.e., GD(1), GD(2), and GD(3)).

e. Total depth of the soil profile (D).

In addition, for estimating three types of water losses, overland flow and sub-surface flow, the following parameters are required:

a. Depth of water table at which evaporative water loss is considered significant (DWTM).

b. Maximum volume of surface storage (VDM).

c. Ratio of evapotranspiration and maximum pan evaporation value (PPAN).

d. Sub-surface discharges through three soil layers Q(1), Q(2), and Q(3).

e. Corresponding storages in these three soil reservoirs SG(1), SG(2), and SG(3).

Finally, routing coefficients to combine flows from three sub-surface layers with the overland flow (i.e. TK(1), TK(2), TK(3), TK(4)) for representative locations in the Kissimmee basin are also necessary along with the assumed number of cascades in layer i (CNR (i)).

Table 1. A typical input basin parameter set used in the sub-basin model.

Basin Parameters	Planning Units 4	2	3	5	6	7
TAS(1)	0	0	0	0	0.0	0
F(1)	0	0	0	0	0.0	0
G(1)	0	0	0	0	0.0	0
GD(1)	0	0	0	0	0.0	0
Q(1)	0	0	0	0	0.0	0
TAS(2)	0	0	0	0	0.0	0
F(2)	0	0	0	0	0.0	0
G(2)	0	0	0	0	0.0	0
GD(2)	0	0	0	0	0.0	0
Q(2)	0	0	0	0	0.0	0
TAS(3)	20.8	22.5	18.5	21.5	22.0	19.5
F(3)	0.0005	0.0005	0	0	0.001	0
G(3)	8.1	8.3	6.8	7.8	9.00	7.8
GD(3)	6.48	5.8	5.5	5.2	7.5	4.9
Q(3)	0.016	0.012	0.0218	0.0145	0.011	0.01
D	48	42	40	40	50.00	41.0
SG(1)	0	0	0	0	0	0
SG(2)	0	0	0	0	0	0
SG(3)	5.35	5.8	5.5	5.2	7.5	5.2
DWTM	27	27	27	27	30	27
YDM	.20	0.15	0.15	0.1	.20	0.10
CNR(1)	0	0	0	0	0	0
TK(1)	0	0	0	0	0	0
CNR(2)	0	0	0	0	0	0
TK(2)	0	0	0	0	0	0
CNR(3)	4	3	5	4	5	3
TK(3)	50	45	27	30	50	54
CNR(4)	2.0	3	3	3	3.	3
TK(4)	25.0	13.0	14	15	30	15

RESULTS, DISCUSSIONS, VERIFICATIONS AND CONCLUSIONS

RESULTS

Using the basic steps outlined in Figure 4A and formulations developed previously, a computer program was developed to take rainfall inputs and to estimate subsurface flow, surface flow, total losses, deep seepage, available storage in soil, storage in depression (at the end of the day), and mean streamflows for 19 planning units for the ten year period (1961-70). These generated values are on a 3 hour basis in tape files (although values can be generated for shorter periods down to 12 minutes) and daily values are compiled.

DISCUSSION

Since huge quantities of data are processed to provide hydrologic information of various types, there is great potential of examining such information from many different angles depending on the task at hand. However, there are certain basic points that are always to be remembered before extending or utilizing such generated hydrologic information to any other beneficial use.

The first important point is that the output of the sub-basin model is the result of a man-made simulation procedure using various mathematical equations representing the hydrologic processes of the physical system. Since the coefficients used in selected formulations are based partially on experiments of local conditions and also on qualitative judgment developed from the practical feel of the region, it is to be cautioned that extensions of these generated values to a new drainage basin with limited available data may lead to erroneous conclusions.

Secondly, the output of the sub-basin model gives a hydrograph (streamflows with time) only at the outlet of each of the 19 drainage basins. These values of streamflows represent the total quantity of water that is contributed by the assumed physical system with its assumed conceptual hydrologic components. These generated values do not provide any information regarding the separate contribution from lake or channel systems.

Thirdly, if a structure is located at the end of a drainage basin, the amount of water that will be allowed to flow through that structure may be quite different than the values given by the sub-basin model because of the fact that the operational characteristics of the water system (i.e., operating schedule of gate operations) is not included in computing these sub-basin streamflows. In other words, the sub-basin model output is to be further processed in the routing procedure to include its distribution through sub-systems (like lakes and channels) and to include operational characteristics of the control structures.

VERIFICATIONS

As reported by Shahane, that although past efforts of Lindahl, Sinha, Storch, Hamrick and Khanal were instrumental in developing a hydrologic model as part of an overall operational watershed for the FCD area, the verification of the model was not pursued to the fullest extent at that point in time; therefore additional verification is warranted.

Although it can be argued that the direct comparison of recorded values through the control structures of the Kissimmee with the simulated streamflows of the sub-basin model is like comparing apples with oranges, the verification task can still suggest in some fashion the critical points which, in turn, show the direction in which further improvements and refinements can be made in the sub-basin model.

With this particular thinking, an effort is made to compare the output of the FCD sub-basin model with the available recorded values. Such verification procedures include:

a. indirect comparisons in terms of correlation coefficients, and

b. direct graphical comparisons with the available historical data.

The methodology of the sub-basin model with its pieces is first applied to the Taylor Creek drainage basin of 100 square miles located on the north side of Lake Okeechobee. Since the hydraulic, hydrologic and agricultural characteristics of the Taylor Creek watershed are well monitored by the ARS of the U.S. Department of Agriculture, and since this drainage area was in its natural form with no control structures to change its natural drainage characteristics during the test period, it was an ideal place to verify and test the FCD sub-basin model. The typical result of such an effort is depicted in Figure 5 (1, 17). Graphical comparisons of mean daily streamflows shown in Figure 5 indicate clearly the adequacy of the sub-basin model and suggest the appropriate choice of coefficients covering the key hydrologic processes.

When the same sub-basin model is applied to the 19 drainage basins (also known as planning units) of the Kissimmee, the streamflows at the mouth of these drainage areas are generated. Due, however, to the controlled nature of the Kissimmee water system, the total quantity of water passing through the control structures can be substantially less or more depending upon the operating schedule of gate openings to control the water levels in the system. As a result, the verification task becomes increasingly difficult because the amount of water contributed by the planning unit (estimated by the sub-basin model) and water released through the structure (if the structure is at the end of the planning unit) are not necessarily the same at any given time for the typical controlled system of the Kissimmee. Among the 19 planning units, only 2 planning units

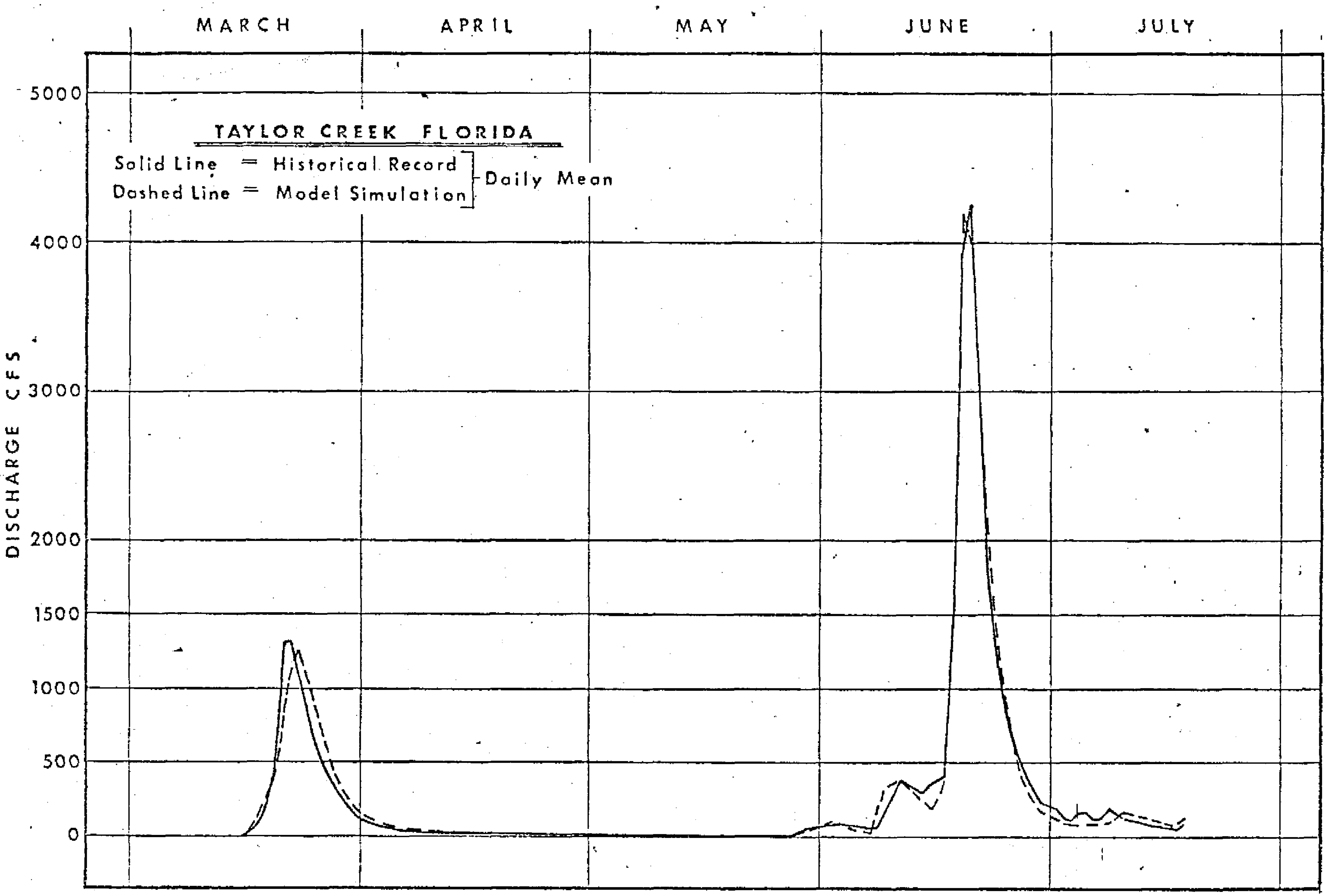


Figure 5. Comparison of simulated and recorded discharge for Taylor Creek (1, 17).

(Boggy Creek basin, No. 4 and Shingle Creek drainage basin, No. 6) are near natural states (with no control structures) having recording stations at the outlet points. Therefore, these planning units (No. 4 and No. 6) are the only available testing sites. In addition, four large areas are also formed by combining several planning units so that total quantity from these large combined basins can be compared with the recorded flows through the corresponding control structure located at the end of the large combined basin. For example, planning units 1,2,3,4,5,6,7,8,9,10, and 11 can be combined to form a large upper Kissimmee basin with S-65 located at one end and thus, recorded flow from the total area of the lower Kissimmee is computed simply by the recorded flow at S-65 for a period of ten years. At the same time, simulated streamflows of planning units, 1,2,3,4,5,6,7,8,9,10 and 11 are added algebraically to obtain simulated streamflows for the lower Kissimmee basin. These two sets are further compared by estimating simple correlation coefficients. This procedure is repeated for the other four combinations with the following planning units and governing control structures.

Table 2

COMBINATION NO.	PLANNING UNITS THAT ARE INCLUDED	UPSTREAM RECORDING STRUCTURE	DOWNSTREAM RECORDING STRUCTURE
4	1, 2, 3, 4, 5, 6, 7	-	S-61
3	1, 2, 3, 4, 5	-	S-59
5	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11	-	S-65
6	All 19 planning units	-	S-65E

Correlation coefficients between simulated values and recorded values for six drainage areas with varying sizes are depicted in Table 2A. The results included in Table 2A suggest the following points:

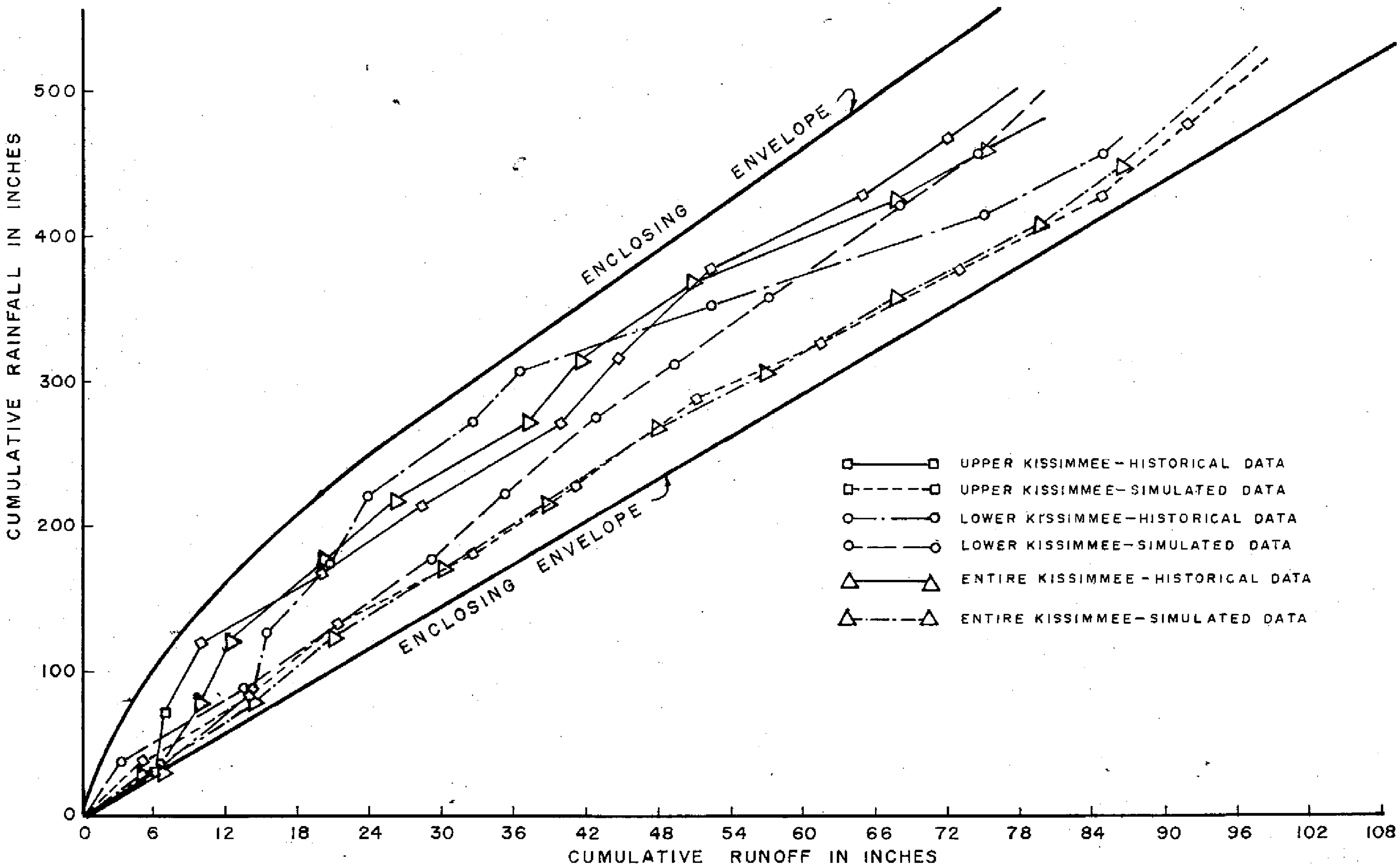
a. The sub-basin model, when applied to individual planning units, can generate realistic streamflows with fairly good correlations in their monthly time series.

b. A simple addition of the outputs from the sub-basin model to obtain simulated flows for combined planning units seems to reduce correlation coefficients partially because time lags of the corresponding streamflows are not accounted for and large planning units tend to become controlled units. This point clearly suggests the need for the development of the routing procedure to be coupled with the sub-basin model output.

c. A slight beneficial averaging effect is observed when the correlation coefficient is improved as the size of the drainage area approaches the total Kissimmee area.

In addition to these two ways of comparing our results with the recorded values, an effort is also made to use the available yearly historical data (with wet and dry period values) compiled by the hydrology division of the FCD. A typical graphical comparison is shown in Figure 6. Useful statistical numbers to facilitate further comparisons are also given in Tables 3 and 4. As mentioned earlier, the main purpose of comparing these two data sets (which are conceptually similar but are different from an operational standpoint) is to pinpoint the missing parameter or hydrologic process (if any) in the sub-basin model. Looking at all comparative tables and figures in light of the above purpose, the following observations can be made:

Period 1960-1970



COMPARISON OF SIMULATED AND RECORDED CUMULATIVE VALUES OF RAINFALL AND RUNOFF FOR UPPER, LOWER AND ENTIRE KISSIMMEE BASIN

FIGURE 6

Table 2A. A Comparison of Simulated (FCD Sub-basin Model Output) and Observed Streamflows in Terms of Correlation Coefficients for Six Different Sized Kissimmee Drainage Basins.

Drainage Area	Size of Drainage Area	Correlation Coefficients
1 *	89.67 sq. miles	0.90
2 **	185.66 sq. miles	0.71
3 ***	298.69 sq. miles	0.67
4 ****	617.12 sq. miles	0.54
5 *****	1134.57 sq. miles	0.63
6 ******	2300 sq. miles	0.70

* Boggy Creek, Planning Unit 4,

** Shingle Creek, Planning Unit 6,

*** Combination No. 3 in Table 2,

**** Combination No. 4 in Table 2,

***** Combination No. 5 in Table 2

****** Entire Kissimmee basin.

Table 3: Correlation coefficients between historical and sub-basin simulated streamflows for lower, upper and entire Kissimmee basin.

Basin	Type of Period	Correlation Coefficients for Rainfall	Correlation Coefficients for Runoff
LOWER	Annual Values	0.8972	0.5027
	Dry Season	0.8122	0.6656
	Wet Season	0.9287	0.5593
	Cumulative Annual Values	0.9998	0.9549
UPPER	Annual Values	0.8622	0.3620
	Dry Season	0.8122	0.6656
	Wet Season	0.9777	0.7613
	Cumulative Annual Values	0.9995	0.9934
ENTIRE	Annual Values	0.9378	0.6440
	Dry Season	0.8333	0.7657
	Wet Season	0.9777	0.7613
	Cumulative Annual Values	0.9998	0.9883

Table 4. "t" values for comparing historical and simulated streamflows for lower, upper and entire Kissimmee basin.

Basin	Type of Period	"t" values for Rainfall	Runoff
LOWER	Annaul values	0.289	-0.189
	Dry Season	+0.469	-1.519
	Wet Season	0.03	0.01
	Cumulative Annual Values	-1.906	-3.135
UPPER	Annual Values	1.839	2.823
	Dry Season	1.38	-1.04
	Wet Season	-1.379	9.91
	Cumulative Annual Values	-1.2662	-6.249
ENTIRE	Annual Values	-0.070	2.823
	Dry Season	1.58	-0.689
	Wet Season	-0.661	5.62
	Cumulative Annual Values	-5.839	-6.472

a. Considering two different techniques of estimating precipitation values for upper, lower and entire Kissimmee, precipitation inputs to the sub-basin model are in agreement with the historical data because of the high observed correlation coefficients (ranging from 0.8122 to 0.999) and "t" values being within the confidence limit of 99.5% indicating that two data sets come from the same normal population. These observed results do indeed increase the confidence in the stochastic methodology of rainfall synthesis used in the sub-basin model.

b. As far as runoff values are concerned, general conclusions are not possible from their comparisons. However, some categorical pecularities can be observed. It is consistently noted from graphical comparisons similar to Fig. 6 that simulated streamflows of the sub-basin model for the wet period for the upper and entire Kissimmee Basin are higher than the historical values. This observation is also reflected in Fig. 6 for cumulative values. Statistically, this particular observation is further substantiated when the corresponding "t" values of the runoff series for the upper and entire basins lie outside the confidence limits of 99.5% suggesting clearly that two data sets do not come from the same population. Due to the nature of the physical reasoning attached to the "coefficient of correlation", the variation in this statistical parameter will not reinforce this observation. Based on this discussion, it seems that the sub-basin model produces an output (for the wet period) which is consistently higher for the upper basin. This particular observation leads to an important conclusion that evaporation from the free surface of the chain of lakes of the upper Kissimmee should be accounted for in the subsequent routing model to improve the output of the sub-basin model. This point is also observed by Dr. Kiker during his previous similar investigations (8).

CONCLUSION CONCERNING SUB-BASIN MODEL

Based on all the possible statistical and graphical comparisons it is observed that the sub-basin model generates hydrologic information which is in general agreement with the recorded values although further improvement is necessary by developing a routing methodology to account for time and spatial distribution of sub-basin streamflows and storage characteristics coupled with evaporation adjustments. At this point, it is anticipated that further refinement in some key basin parameters of sub-basin model may be necessary after all the pieces (such as sub-basin model, routing model, hydraulic formulations and computer programs) are put together and after realistic routed values for upper and lower Kissimmee basin are obtained.

ROUTING MODEL

PURPOSES

In technical terms, routing is defined as a procedure to determine the time and spatial distribution of streamflows or a flood wave at a point in a water system by considering the hydraulic and hydrologic data at one or more points upstream (3). In our specific investigations, the basic purposes of developing routing methodology are:

1. To distribute sub-basin model output through the system of the lake channel and controlling structures,
2. To combine stage-storage fluctuations of lake with the stage-discharge characteristics of the channel sections for developing a joint methodology of reservoir and channel routing,
3. To include operational characteristics of the controlling gates coupled with the routed simulated stages for estimating discharges through various controlling structures,
4. To improve sub-basin model output by including the key process (if any) of the lake or channel which might be excluded from the assumed conceptual physical system, and
5. To provide the basis for examining the effects of changing operational parameters on the hydrologic characteristics of the Kissimmee water system with complete independence from the analysis of the historical data.

FACTORS FOR THE SELECTION OF THE ROUTING METHODOLOGY

It is to be emphasized at this stage that the conventional techniques are mostly developed for a natural system with historical data to refine the model. Therefore, when operational characteristics of the controlled system are to be incorporated into the overall methodology, a different routing procedure is warranted because the coefficients or parameters used in the conventional methods may vary substantially with the different gate operations. As a result, routing procedure is to be designed to include directly or indirectly the operational data coupled with simulated stages rather than the historical data. In other words, due to our specific requirement of developing an operational watershed model, it seems necessary to modify the available routing methods.

Another factor for developing a modified routing methodology relates to the partially controlled nature of the Kissimmee basin. Again, since our task is to distribute sub-basin model outputs through lakes, channels and controlling structures it is essential to design a routing methodology to link these component parts together in some fashion using the operating schedule (if necessary).

While routing the streamflows through the controlled Kissimmee water system, the concept of "time lag" is an important factor to be considered in the routing techniques. In conventional methods, the estimated time lag is based on the travel time of the water system. However, for the water system of the Kissimmee with typical lakes, channels and controlling points it may not be possible to compute the travel time since it will vary considerably as a function of gate openings. This again looks forward to a routing methodology (an entirely different or modified form of conventional routing methods) for including the possible time-lag in streamflow as it moves through lakes, channels and controlling structures.

Another key factor in any routing methodology is the routing period. If the routing period is sufficiently short, then hydrographs will be presented more adequately. However, by decreasing the routing period, data collection steps becomes more laborious because of the increased sample size. As the general rule, it is recommended that the routing period should be equal to or somewhat shorter than the travel time of the flow through the reach and, as a lower bound, it should be short enough so that the hydrograph during that period approximates a straight line (3). Considering these points, it was decided to select a 3 hour period in our routing methodology. To keep data collection steps manageable, it was also decided to demonstrate the routing procedure for a period of 1 year (1970).

INPUT INFORMATION AND ESSENTIAL FORMULATIONS

INPUT INFORMATION

Since we are trying to demonstrate the routing model for a one year period of 1970, it is necessary to obtain hydrologic base-line information just before this period in upper and lower Kissimmee lake, channel and controlling structures. Based on the requirements of our routing procedure, the initial recorded stages of all the lakes of the upper Kissimmee and discharges with tailwater, headwater elevations at all the structures of the Kissimmee basin are necessary to start the routing model. Such information (also known as initial conditions) is compiled in Tables 5, 6 and 7. Based on Table 6, weighing factors are further computed using maximum storage, average surface area of the lake and maximum surface area for each of the lakes contained in a particular planning unit. Such proportioning factors are given in Table 8. These factors are used in distributing local inflows (sub-basin model output) in the corresponding lakes of a particular planning unit. Three sets of these proportioning factors are intentionally prepared with a view to perform sensitivity analysis in the later stage (if necessary).

As a result of the comparisons of sub-basin output with historical data, it is concluded that evaporation values in the upper Kissimmee lakes should be included in the routing procedure to improve the sub-basin outputs and thus their subsequent spatial and time distribution. With this intention, monthly

Table 5. Recorded stages of upper Kissimmee Lakes on December 31, 1969.

LAKES	STAGES
Pierce	77.54
Tohopekaliga	55.05
East Tohopekaliga	57.98
Alligator and Brick	63.92
Mary Jane	60.93
Coon	63.92
Hart	60.92
Cypress	52.54
Kissimmee	52.25
Gentry	61.78
Tiger	52.28
Hatchineha	52.25
Myrtle	63.40
Preston	63.40
Trout	63.92
Weohyakapka	62.18
Joel	63.40
Lizzie	63.92
Center	63.92

Table 6. Surface Areas at Maximum Stages of Different Lakes

LAKE	AVERAGE SURFACE AREA IN ACRES	MAXIMUM CAPACITY IN ACRE-FT	MAXIMUM AREA IN ACRES
Pierce	2,251	3,514	3,592
East L. Toho.	12,300	250,000	19,980
Allig. & Brick	3,750	79,500	11,260
Tohopekaliga	21,200	420,000	30,700 at stage 61
Mary Jane	725	16,700	3,100
Hart	253	16,700	7,069 at stage 58
Trout	197	7,215	750 at stage 65
Coon	65	3,663	225 at stage 64
Myrtle	365	5,750	1,290
Lizzie	240	18,870	1,220 at stage 65
Preston	515	7,990	1,455
Cypress	2,600	55,000	4,011
Gentry	1,730	48,300	9,600
Kissimmee	33,500	716,000	100,000
Rosalie	5,540	100,000	9,130
Tiger	3,060	42,800	4,400
Weohyakapka	7,280	95,000	11,000
Alligator	1,350	80,500	9,170
Hatchineha	4,000	43,396	9,439
Joel	124.6	2,716 at stage 65	219.4 at stage 60
Center	365	19,200	4,975

Table 7. A set of Initial Conditions of Structures Useful in the Routing Model for the Upper and Lower Kissimmee Basin.

Structure	TWE*	HWE*
S-57	61.01	63.40
S-58	63.29	63.89
S-59	55.70	57.58
S-60	61.90	63.80
S-61	52.56	54.71
S-62	58.12	61.13
S-63	56.75	62.00
S-63A	51.83	56.49
S-65	46.60	52.34
S-65A	40.66	46.46
S-65B	33.96	40.62
S-65C	27.03	33.96
S-65D	20.86	26.92
S-65E	13.27	20.79

* These tailwater and headwater elevation (TWEaand HWE) values are at the end of December 31, 1969 which is the last previous day of the time period (January 1, 1970 to December 1970) considered in our routing model.

Table 8. Proportioning Factors for the Lakes of the Upper Kissimmee to Distribute the Local Inflows from Appropriate Planning Units

		Proportioning Factors		
Lake	Maximum Storage in acre/ft.	Based on Surface Area at Max. Stage	Average Surface Area	Corresponding Planning unit of the available local flows
Alligator & Brick	0.52	0.610960	0.812216	1
Lizzie	0.08	0.066196	0.051982	1
Coon	0.31	0.282108	0.093134	1
Trout	0.09	0.040695	0.042668	1
Joel	0.09	0.074012	0.124029	2
Myrtle	0.91	0.925988	0.875971	2
Mary Jane	0.500000	0.304848	0.741309	3
Hart	0.500000	0.695152	0.258691	3
East Toho.	1.000000	1.000000	1.000000	4,5
Toho.	1.000000	1.000000	1.000000	6,7
*Cypress	0.342485	0.298216	0.296015	8,10,11
*Hatch.	0.657515	0.701784	0.696605	8,10,11
Gentry	1.000000	1.000000	1.000000	9
Kissimmee	1.000000	1.000000	1.000000	12,13,14

*Based on the manual assessment of drainage areas of Cypress and Hatchineha, the following proportioning factors are estimated for these Lakes:

Cypress	0.34	-	-	10
Cypress	0.40	-	-	8
Hatch.	1.00	-	-	11
Hatch.	0.66	-	-	10
Hatch.	0.60	-	-	8

pan evaporation values recorded at Orlando Weather Bureau station are taken and various constants to convert these monthly values to 3 hour lake evaporation values are computed. Since lake evaporation is a function of stage, final equations to estimate 3 hour evaporation values for the lake chain of the upper Kissimmee are given in Table 9.

ESSENTIAL FORMULATIONS

As an essential part of the simulation procedure, our methodology also depends heavily on the formulations of various water systems. Basic forms of the equations which are used in our analysis are summarized in Table 10. As shown in this table, formulations are classified according to the type of system (i.e. lake, channel or controlling structures). They are described below.

FORMULATIONS FOR LAKE SYSTEM

Essentially, the parameters which are of interest to our simulation analysis are stages, storages, inflows and outflows for various lakes. The first two equations of the lake system given in Table 10 tie together, change in storage (ΔS) and changes in stage to the characteristics of inflow, outflow and initial stages. These equations are simple forms of mass-balance equations. In addition, it is also necessary to know the stage-storage relationships for all the lakes of the upper Kissimmee. These relationships can be in either tabular form or in the mathematical form. Although it is clear that the use of tabular form similar to Table 11 is the most accurate way of converting back and forth the stages and storages of the lake, the tabulated values may consume more memory of the computer. Based on this thinking, nonlinear exponential relationships established by the multi-variate program are used in our routing methodology. Among the nonlinear exponential functions and polynomial equations, the exponential functions are selected due to the high correlation coefficients associated with them and

Table 9. Pan Evaporation Values and Associated Equations with Coefficients for Estimating Evaporation of Lakes of Upper Kissimmee Basin.

Month	Pan Evaporation* in Inches (A)	Coefficients (K) for Converting to 3 hr. values (B)	Equations for Estimating Lake Evaporation in Acre-ft.
January	2.636250	0.0002688	$(A)\cdot(B)\cdot(X_{i1})$**
February	3.400000	0.0002976	$(A)\cdot(B)\cdot(X_{i2})$
March	4.925000	0.0002688	$(A)\cdot(B)\cdot(X_{i3})$
April	6.188750	0.0002778	$(A)\cdot(B)\cdot(X_{i4})$
May	7.292500	0.0002688	$(A)\cdot(B)\cdot(X_{i5})$
June	6.741429	0.0002778	$(A)\cdot(B)\cdot(X_{i6})$
July	6.573333	0.0002688	$(A)\cdot(B)\cdot(X_{i7})$
August	6.298750	0.0002688	$(A)\cdot(B)\cdot(X_{i8})$
September	5.337500	0.0002778	$(A)\cdot(B)\cdot(X_{i9})$
October	4.270000	0.0002688	$(A)\cdot(B)\cdot(X_{i10})$
November	3.047778	0.0002778	$(A)\cdot(B)\cdot(X_{i11})$
December	2.276667	0.0002688	$(A)\cdot(B)\cdot(X_{i12})$

* Data from Orlando Station.

** X_{ij} = Lake surface in acres for ith lake in jth month, ($i = 1, \ldots 14$ and $j = 1, \ldots 12$)

Table 10. Basic Forms of Equations Useful in the Model* (16, 17)

System	Formulations	
Lake System	1. $(\text{stage})_{t+1} = (\text{stage})_t + (\Delta S)_{t+1}$	(A)
	2. $(\Delta S)_{t+1} = I_{t+1} - O_{t+1}$	(B)
	3. $WSE = a\ (S)^b$	(C)
	4. polynomial equations $\text{stage} = A_0 + A_1(S) + A_2(S)^2 + A_3(S)^3 + A_4(S)^4$	(D)
Channel System	1. $\frac{dy}{dx} = \frac{SO - SE}{1 - \frac{\alpha Q^2 T}{gA3}} = \dot{y}$	(E)
	2. $SE = \frac{(n)^2 V2}{2.22\ (H.R)^{4/3}}$	(F)
	$SE = \frac{n^2 Q^2 P^{4/3}}{2.2\ A^{10/3}}$	
	3. $Y_{i+1} = Y_i + [\frac{\dot{Y}_{i+1} + \dot{Y}_i}{2}]\ DX$	(G)
Structures Operations	1. $Q(N) = P(GO)^r\ (EH)^S$	(H)

*Notations are explained at the end of the report.

Table 11. The typical stage-storage values for the lakes of upper Kissimmee basin.

Stage ft. m.s.l. Alligator - Brick	Storage Acre-ft	Stage ft.m.s.l. Lake Lizzie	Storage Acre-ft	Stage ft. m.s.l. Lake Coon	Storage Acre-ft
54	10,800	41	0	49	5
55	12,600	42	30	50	13
56	14,800	43	50	51	27
57	17,400	44	100	52	45
58	20,300	45	170	53	70
59	23,500	46	250	54	104
60	27,000	47	340	55	150
61	30,600	48	425	56	210
62	34,500	49	560	57	282
63	38,400	50	665	58	370
64	43,300	51	800	59	470
65	50,200	52	990	60	570
66	59,000	53	1,210	61	698
67	68,700	54	1,475	62	830
68	79,500	55	1,760	63	985
		56	2,135	64	1,140
		57	2,500	65	1,476
		58	2,950	66	1,925
		59	3,435	67	2,660
		60	4,150	68	3,663
		61	4,950		
		62	5,700		
		63	6,500		
		64	7,350		
		65	8,350		
		66	10,450		
		67	16,085		
		68	18,870		

their use by the previous investigators; however, if at the later stage they are found to be inadequate in estimating stages within the accepted accuracies, then we will still have the option to use tabulated values directly.

FORMULATIONS FOR CONTROLLING STRUCTURES

The operational characteristics of the Kissimmee water systems are reflected in the formulations of the controlling structures. Variables considered in these formulations are gate openings (GO), headwater elevation (HWE), and tailwater elevation (TWE) with discharge as a dependent variable as shown by Equation 1 for structure operations in Table 10. In our routing methodology these equations are used to compute the discharge through the structure knowing the simulated tailwater and headwater stages for a given set of gate openings.

CHANNEL FORMULATIONS

The development of the channel formulations and using them in a convenient fashion in routing methodology are some of the steps that make our procedure different than previously attempted techniques. Essentially, the hydraulic formulations given in Table 10 for the channel system relate to:

1. A differential equation representing gradual varied flow with slope of energy line, channel bottom slope, discharges, cross-sectional area, top width of the channel and velocity head coefficients as variables and rate of change of depth (with distance) as a dependent variable (Equation 1 of Table 10).
2. Manning's equation combining hydraulic characteristics of the flow (i.e., velocity, Manning's coefficients, slope of energy line) with the physical characteristics of the channel cross-sections such as cross-sectional area (A) and perimeter (P). (Equation 2 of Table 10 of channel system).

3. An iterative equation based on a numerical integration technique of trapezoidal rule applied by Prasad (13, 14) to estimate the water depth (and then water surface elevation) at the end of the channel section (Equation 3 of Table 10 of the channel system).

In other words, for channel routing, we are using Manning's equation as against different existing routing methods that are presented in the earlier studies. The obvious reasons for such selection are as follows:

1. Unlike other methods, Manning's equation does not depend heavily on many coefficients which are mostly determined from historical data.
2. The channel cross-sections are built into that equation to represent realistic flow conditions.
3. An adequate sensitivity analysis has been done previously to get realistic values for Manning's coefficient "n" for the upper as well as the lower Kissimmee basins (11, 16, 17).
4. A numerical technique is readily available to use iterative procedures for computing water stages,
5. It is also possible to include marsh area of the lower Kissimmee with a different formulation for Manning's coefficient in some parts of the cross-sectional channel data.

Originally, we had planned to use Manning's equation with Prasad's iterative procedure as an intermediate step in our over-all routing methodology. However, realizing the computer time involved in the synthesis of channel data, subsequent iteration procedure and its hold-up effects on other steps of the routing procedure, it was decided to use the existing FCD backwater program separately and then transfer the results independently to the required point in the main program. This task can be achieved in two ways. In the first method, the main program can sepcify the values of Q and their distribution pattern with other essential computational parameters and then the FCD backwater program can be called in as a subroutine. The output of the subroutine can then be transferred to the main program where it will be processed further as required by the logic of the main program. In the second approach, for a given set of discharges and stages, the FCD backwater program can be run independently to perform backwater computations for all the channel sections of the Kissimmee using the available channel cross-sectional data. For a given channel section, this program can generate a set of upstream, downstream stages along with discharges and storages. Using this data set, empirical relationships based on statistical principles can be derived for these variables. These established mathematical relationships (also known as backwater functions) are then used in the main program to replace directly the backwater computational steps. Between these two approaches, the second approach seems to be more convenient in short term as well as long term basis, and thus, it is used in our routing methodology.

As a first step toward developing these backwater functions, the ranges of discharges and stages are to be decided for each of the channel sections. After reviewing the general design memorandums for the channel systems of the Kissimmee, the upper and lower limits of discharges and stages were selected. Such ranges are given in Table 12. For each channel section, the corresponding discharge and stage ranges are divided into about ten equal parts and for each set of discharge and downstream stage, backwater program provides the upstream stage when cross-sectional data is arranged from the downstream to the upstream side. A stepwise procedure for arriving at backwater functions includes:

1. Processing of existing cross-sectional data for all the channels of the Kissimmee.

Table 12. Ranges of Discharges and Stages Used in FCD Backwater Program for Different Channel Sections

Channel Section	Ranges of Discharge in cfs	Ranges of Stages in ft. msl
C-38 between S-65 and S-65A	0 - 13000	44 - 55
C-38 between S-65A and S-65B	10 - 18000	35 - 45
C-38 between S-65B and S-65C	0 - 20000	25 - 36
C-38 between S-65C and S-65D	0 - 25000	25 - 30
C-38 between S-65D and S-65E	0 - 30000	18 - 23
C-37	0 - 7000	45 - 60
C-36	0 - 4000	45 - 60
C-35	0 - 3000	45 - 60
C-34	0 - 2000	45 - 65
C-33	0 - 400	55 - 70
C-32	0 - 200	55 - 75
C-31 below S-59	0 - 700	45 - 65
C-30 below S-57	0 - 300	50 - 70
C-30 above S-57	0 - 300	50 - 70
C-29 above S-62	0 - 250	55 - 70
C-29 below S-62	0 - 600	50 - 65

2. Arranging the cross-sectional data to the correct input format specified for the existing FCD program of E070A for channel sectional analysis,
3. Using E081A program for backwater computations,
4. Getting punched output from E081A program relating downstream stage, upstream stage, average discharge and computed storage, and
5. Using such punched output in the "multivariate regression analysis" subroutine (E069) for developing backwater functions for three or four variables of the channel section.

The results from the FCD backwater program (a combination of E070A and E081A) for all the channel sections of upper and lower Kissimmee are compiled in several files which are stored in the Water Planning Division. These files constitute several hundred pages and contain:

1. Cross-sectional data used in the backwater program,
2. Water stages, water depth, top width, conveyance, accumulated surface area, accumulated volume, right and left intercepts of channel cross-sections for each of the stations considered in a particular channel section.

To develop the mathematical relationships between upstream stage, downstream stage, storage and discharges, it is necessary to first look into the nature of the functional relationships to be attempted. To proceed in this direction, the discharge values, the difference between upstream and downstream stages (ΔH) and downstream stages are plotted for C-38A (C-38 between S-65 and S-65), C-38E (C-38 between S-65D and S-65E), C-35 below S-61 and C-29A above S-62. These graphs, coupled with the requirements of the existing regression program (E069), suggest that the nonlinear relationships between ΔH, Q and D.S.S. apply more conveniently to the channel sections of lower Kissimmee than for the upper

Kissimmee channels. As a result, the existing multivariate regression subroutine is extensively used to try different combinations of four variables (U.S.S., D.S.S., discharge and storage) for lower Kissimmee sections and different combinations of three variables (U.S.S., D.S.S. and discharge) for the upper Kissimmee channels.

Since the storage characteristics of the upper Kissimmee units are largely related to lake storage with relatively insignificant storage in the channel sections of the upper Kissimmee basin, linear and nonlinear relationships are developed for these channels with three variables of upstream downstream stages and discharges. Whereas for five pools of the lower Kissimmee, channel storage being significant, storage parameter is included as the fourth variable.

After examining the statistical coefficients of various feasible formulations (14, 15), the selected equations for channel sections of the upper and lower Kissimmee are given in Tables 13 to 17.

COMPUTATIONAL METHODOLOGY

After developing and refining various pieces presented earlier, the next important step is to link them together to distribute the sub-basin flows through the lake, channel and controlling structures of the Kissimmee basin. Considering the interactions between local inflows, discharge through the connecting channels of two lakes and discharge through the controlling structures, there can be several variations of iterative routing procedures that can be developed to achieve the same objective.

Two illustrations to explain the selected methodology (one for Alligator-Brick system in upper Kissimmee and the other for Pool A in lower Kissimmee) are presented with associated routing steps. For a better understanding of the following steps, the reader is advised to refer simultaneously to Figures 1 and 2.

Table 13. Nonlinear formulations of discharges for the typical seven channel sections of the upper Kissimmee basin.

Channel Section	Nonlinear Relationship $Q = (US - DS)^A \, (DS)^B$		
	A	B	r^2
C-32G	0.19562817	1.37327452	0.99036109
C-32B	0.12933563	1.31192007	0.99028838
C-32D	0.11312801	1.28232715	0.98835559
C-32F	0.04781795	1.17262063	0.97613564
C-29	0.23189354	1.53817995	0.99206125
C-37	0.44362025	2.25302023	0.99901088
C-36	0.40565648	2.17679705	0.99862609

r = correlation coefficient

Q = mean discharge,

US = upsteam,

DS = downstream

Table 14. Nonlinear Formualtions for the Channel Sections of Upper Kissimmee basin.

Channel Section	Nonlinear Relationship U.S.S. = $(Q)^A$ $(D.S.S.)^B$ A	B	r^2
C-29	0.005731	0.993423	0.999993
C-29A above S-62	0.000190	0.999820	1.000000
C-29A below S-62	0.002290	0.997502	0.999999
C-29B	0.005924	0.992899	0.999976
C-30 above S-57	0.0037	0.99969	1.000000
C-30 below S-57	0.000792	0.999316	1.000000
C-31 above S-59	0.007304	0.990799	0.999986
C-31 below S-59	0.003198	0.992627	0.999998
C-32B	0.000350	0.999690	1.000000
C-32C above S-58	0.000696	0.999392	1.000000
C-32C below S-58	0.00001	1.000050	1.000000
C-32D	0.00014	0.99988	1.000000
C-32F	0.006202	0.994513	0.999973
C-32G	0.000077	0.999936	1.000000
C-33 above S-60	0.000916	0.999313	0.999999
C-33 below S-60	0.002472	0.997548	0.999998
C-34 between S-63 and S-63A	0.007898	0.992503	0.999944
C-34 between S-63A and Lake Cypress	0.004327	0.993701	0.999997
C-35	0.006335	0.989862	0.999998
C-36	0.003194	0.994645	0.999999
C-37	0.007644	0.986547	0.999995

r = correlation coefficient

Q = mean discharge

U.S.S. = upstream stage,

D.S.S. = downstream stage

$Q > 0$

Table 15. Stage-storage-discharge relationships for the lower Kissimmee basin.

Channel Sections	Nonlinear Relationship $US = (DS)^A(\log Q)^B$ A	B	r^2
C-38A	0.93525809	0.12357836	0.99999427
C-38B	0.80300638	0.34915258	0.99997801
C-38C	0.72539726	0.45337676	0.99993335
C-38D	0.72979747	0.42254163	0.99995117
C-38E	0.84436366	0.22342889	0.99995183

r = correlation coefficient

US = upstream stage,

DS = downstream stage

Q = mean discharge

Q > 0

Table 16. Stage-storage-discharge relationships for the lower Kissimmee basin.

Channel Sections	Nonlinear Relationship $DS = (\log Q)^A (\log ST)^B$ A	B	r^2
C-38A	0.46661167	1.29225620	0.99974083
C-38B	0.06123664	1.59817418	0.99994493
C-38C	-0.11387716	1.70097296	0.99985066
C-38D	-0.32464046	1.79672855	0.99987939
C-38E	-0.31844141	1.68094907	0.99921370

r = correlation coefficient

DS = downstream stage

Q = discharge

ST = storage in acre ft.

$Q > 0$

Table 17. Stage-storage-discharge relationships for the lower Kissimmee basin.

Channel Sections	Nonlinear Relationship Storage = $e^{(\log DS)^A (\log Q)^B}$ A	B	r^2
C-38A	1.94349507	-0.18936090	0.99940581
C-38B	1.75866251	-0.03262747	0.99987813
C-38C	1.65939267	0.05420344	0.99987124
C-38D	1.54070909	0.17934023	0.99995894
C-38E	1.21042069	0.39086445	0.99968979

r = correlation coefficient

DS = stage

Q = discharge

e = 2.718281828

Q > 0

Consider the first three lakes systems with Lakes Alligator and Brick in the middle and Lakes Gentry and Lizzie on each side. Channel sections associated with this system are:

1. C-32G between Lakes Alligator and Lizzie,
2. C-33 between Lakes Gentry and Alligator,

With the initial stage recorded on December 31, 1969 for the Alligator-Brick system, initial storage is computed from the stage-storage tables for the Alligator-Brick lake.

Using initial recorded stages at Lake Lizzie and at Alligator, the initial discharge is estimated by channel formulations given in Table 13 (I_2).

Since, for channel section C-33, there is a structure (S-60) located between Gentry and Alligator, the initial discharge through the structure is estimated from the recorded tailwater and headwater elevations (TWE and HWE) and the first 3 hour gate opening data using the discharge rating curves of controlling structures.

Now by considering the first 3 hour local inflow of January 1, 1970, (which is a fraction of the corresponding sub-basin model output according to Table 8, (I_1)) initial estimate of the discharges through C-33 (which is assumed to be the same as the discharge through S-60) and initial estimate of the discharge through C-32G, the change in storage (ΔS) in Lake Alligator-Brick system is computed as follows:

$$\Delta S = I_1 \pm I_2 \pm O_1$$

where

I_1 = a proportion of 3 hour simulated streamflow from the FCD sub-basin model for planning unit 1,

I_2 = initial estimate of flow between Lakes Alligator and Lizzie,

O_1 = initial estimate of flow between S-60 and Lake Alligator-Brick system through C-33.

After converting this ΔS value to acre-ft., new storage is obtained by adding ΔS to the initial storage. Corresponding to this new storage for the first 3 hour period, new stage is computed from the corresponding stage-storage, Table No. 11.

Using this new stage at the end of the 3 hours and using initial stages at Lake Lizzie and headwater elevation at S-60, new discharges through C-33 and C-32G are estimated.

Using these new discharges and the same value of local inflow into Lake Alligator, the ΔS is again computed. If the absolute difference between new and previous estimates of ΔS is within the storage error margin, then iteration is stopped for Lake Alligator and new stages and new estimates of discharges through C-33 and C-32G become final estimates which are subsequently used in the next step for Lake Lizzie.

If the new estimate of ΔS is significantly different than its previous values, then that means that estimated discharge values are to be modified. This is done by taking averages of the new and previous values of discharges through C-33 and C-32G. With these average values of discharges, ΔS is again computed for the Alligator and Brick system. This procedure is continued until the difference between previous and new estimates of ΔS is within the prescribed limit.

From the final estimate of discharge through C-33 and final 3 hour stage of Alligator Lake, headwater elevation (HWE) at S-60 is computed using the equations of Table 14. This stage is the simulated headwater stage which, in turn, is used to estimate discharge in the next time step.

The final outcome of the iterative procedure for this three lake system gives us:

1. Final estimate of stage in the Lake Alligator-Brick system at the end of the particular 3 hour period considered.
2. Final values of discharges through C-33 and C-32G at the end of the 3 hour period.
3. Simulated headwater stage at S-60.

Thus, after completing the iterative procedure of this three lake system, the next three lake system is considered and the procedure is repeated for all the lakes of the upper Kissimmee.

After completing the iterative steps of the upper Kissimme for a given time step of 3 hours, five pools of the lower Kissimmee are then considered. Due to the storage in the channel as well as in the surrounding vegetative marsh areas, the iterative procedure for these pools is different than the lake-channel system of the upper Kissimmee. Such procedure is illustrated step by step for Pool A which is basically the channel C-38 between S-65 and S-65A and the contributing surrounding area of planning unit 15.

Using initial tailwater and headwater (TWE and HWE) stages at S-65 and S-65A with the corresponding first 3 hour gate operations at these structures, initial discharges through S-65 and S-65A are estimated from the given rating curves.

Taking the average value of these two initial discharges through S-65 and S-65A and recorded initial HWE at S-65A, initial storage is estimated for Pool A from the equation in Table 17.

Now for the first 3 hour period of January 1, 1970, the change in storage for this pool A is determined by

$$\Delta S = I_1 + I_2 - O_1$$

where

I_1 = final estimate of discharge through S-65 (This is obtained from the application of the same procedure to Lake Kissimmee).

I_2 = 3 hour streamflow values for planning unit 15 obtained from the sub-basin model,

O_1 = discharge through S-65A (This can be either estimated from the rating curve).

Using this new storage (i.e. initial storage + ΔS) and average discharge (i.e. $Q = \frac{I_1 + O_1}{2}$) upstream and downstream stages in C-38A are estimated from the equations given in Tables 15 and 16.

Thus, when tailwater elevation at S-65 and headwater elevation at S-65A are estimated, a new discharge corresponding to these new simulated stages is then obtained. Using these values, the change in storage is recomputed. If the new value of the storage is not significantly different than the previous value, then iteration is stopped.

If the new value of the storage is significantly different than the previous value, then averages of new and previous discharges at S-65 and S-65A are taken and iteration is continued until the new value of storage is similar to the previous estimate within the prescribed storage error margin.

Thus, the outcome of this iteration procedure as applied to pool A gives us:

1. 3 hour simulated headwater elevation at S-65A and
2. 3 hour simulated tailwater elevation at S-65.

This procedure is continued for the other four channel sections and similar analysis is performed for the next time step using simulated discharges and stages of the previous time step.

CONCLUSIONS CONCERNING ROUTING METHODOLOGY

1. The routing methodology (which is demonstrated only for one year of 1970 on a 3 hour basis) is designed to take into account:
   a. The change in the flow direction due to the prevailing operating rules at controlling structures,
   b. Indirectly the possible time lag between the streamflows of the sub-basin model, and
   c. The interactions of stage-storage and discharge characteristics of lakes and channels of the controlled water system of the upper and lower Kissimmee basin.
2. While designing and developing different stages of our methodology many simplifications in the form of assumptions, approximations and speculations are made. They are as follows:
   a. Lake Brick and Alligator are considered as one combined lake,
   b. Lake Ajay is also treated as combined with East Tohopekaliga Lake,
   c. Since the water level in Lake Myrtle and in Preston is the same, Lake Preston is not included in the analysis although its share in distributing sub-basin model output is indirectly accounted for,
   d. Similarly, offline Lake Center is not included in the analysis since the water level of Lake Coon and Lake Center is the same,
   e. Because of the short and straight forward nature of channels C-29B, C-34 above S-63, C-35 above S-61, they are not included in the backwater computational methodology. For these channel sections it is assumed that the water level of the nearest lake is equal to the appropriate headwater or tailwater elevation at the structures of these channels,

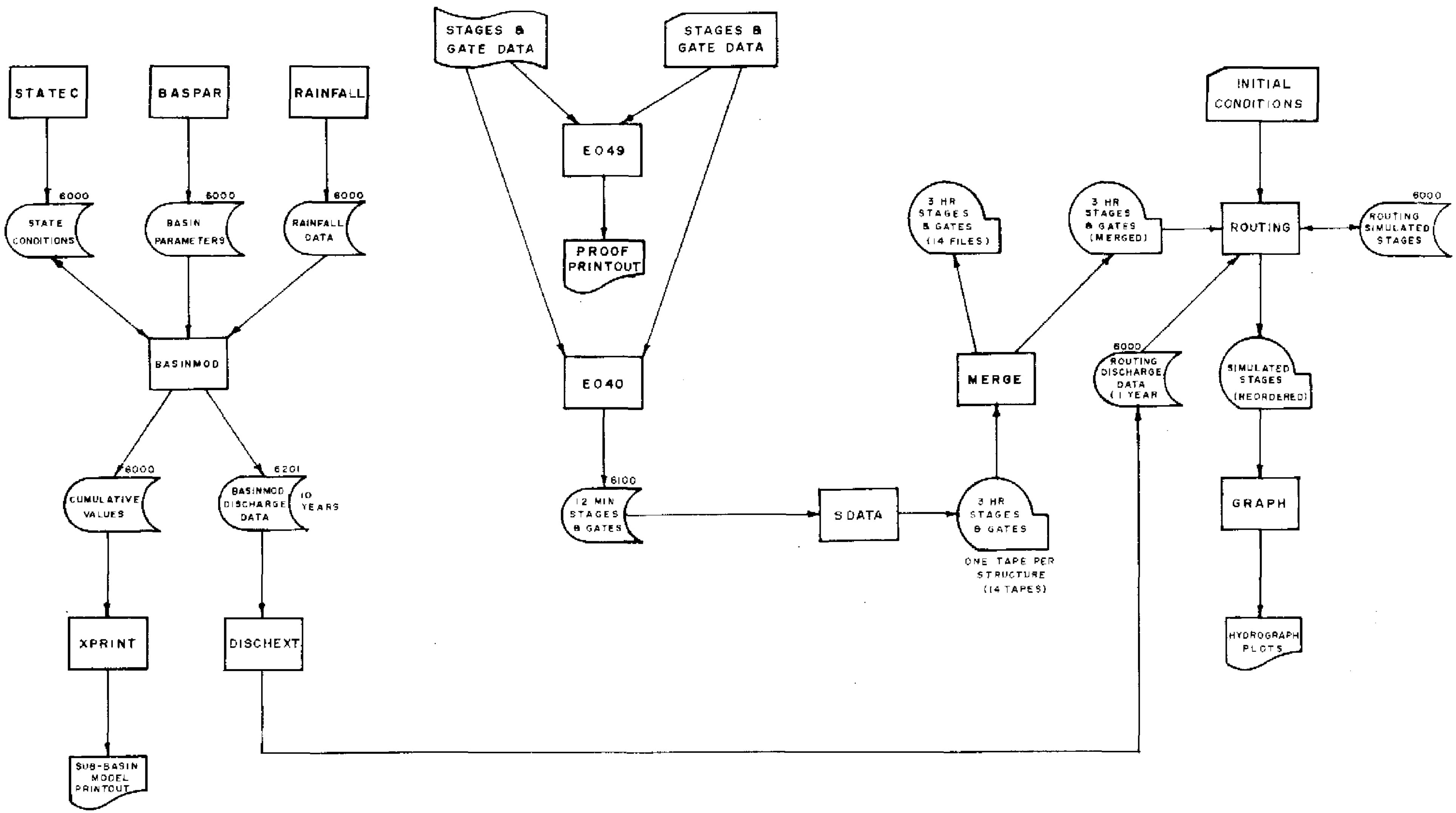
3. Since the routing methodology and associated program design is intended to be general, the operational characteristics of the controlled system of the Kissimmee is adequately built into the steps of our routing procedure. In addition, this procedure being susceptible to the parametric sensitivity analysis, it is possible to examine the effect of changed conditions on the different parameters under investigation.

OUTCOME OF THE COMBINED PROGRAM

NATURE OF THE OUTPUT

After refining all the previously described pieces of the water quantity model shown in figure 7 the output is essentially the net result of the interactions of various subcomponents of such hydraulic simulation procedure. Since, as per our primary goal, we are able to develop a framework for combining sub-basin model with routing model for the entire Kissimmee basin, the primary output from such methodology consists of

1. 3 hrs. simulated discharges through all the channel sections of the upper and lower Kissimmee for the year 1970.
2. 3 hrs. simulated mean discharges through all the control structures for the full year of 1970.
3. 3 hrs. simulated stages for all the major 14 lakes of the upper Kissimmee basin.
4. 3 hrs. simulated tailwater and headwater stages at all the control-structures of the upper and lower Kissimmee basins.
5. Storages in all the major lakes and storages for five sections of the lower Kissimmee at the end of every 3 hrs. for the entire year of 1970.
6. Comparative tables for assessing the adequacy of the output by comparing recorded TWE and HWE with simulated values at nine controlling structures (S-58, S-57, S-62, S-59, S-61, S-63, S-60, S-65 and S-65E).
7. Graphical comparisons of simulated stages and recorded stages of eight lakes of the upper Kissimmee basin.
8. Graphical comparisons of the simulated and recorded daily discharges through S-59, S-63, S-57, S-62, S-60 and S-65.



SYSTEM CHART OF THE OVERALL WATER QUANTITY MODEL

FIGURE 7

Excluding the time required for preparing manually the comparative graphs, it takes about 5 hrs. on the CDC 3100 computer to obtain the above required output. Further break-down of these 5 hours is as follows:

1. Sub-basin output (139 minutes)
   a. state conditions program (2 minutes)
   b. BASIN parameters (2 minutes)
   c. Rainfall decomposition (20 minutes)
   d. BASIN model (100 minutes)
   e. printing the result (15 minutes)
2. Routing model output (150 minutes)
   a. DISCHEXT program (15 minutes)
   b. ROUTING program (120 minutes)
   c. GRAPH program (15 minutes)

In addition to the above primary output, a tremendous amount of secondary output is also generated. Categorically, such secondary output consists of

1. a complete set of cross-sectional data of all the channel sections of the upper and lower Kissimmee basin in computer usable form.
2. the backwater computations at every available cross-section of all the channel sections of the upper and lower Kissimmee for different ranges of discharge and stages as shown in Table 12.
3. various types of backwater functions correlating the downstream stage, upstream stage and discharge for the channel sections of the upper Kissimmee and correlating four variables (US, DS, Q and storage) for the five sections of the lower Kissimmee.
4. linear-nonlinear stage-storage functional relationships for all the 14 lakes of the upper Kissimmee.

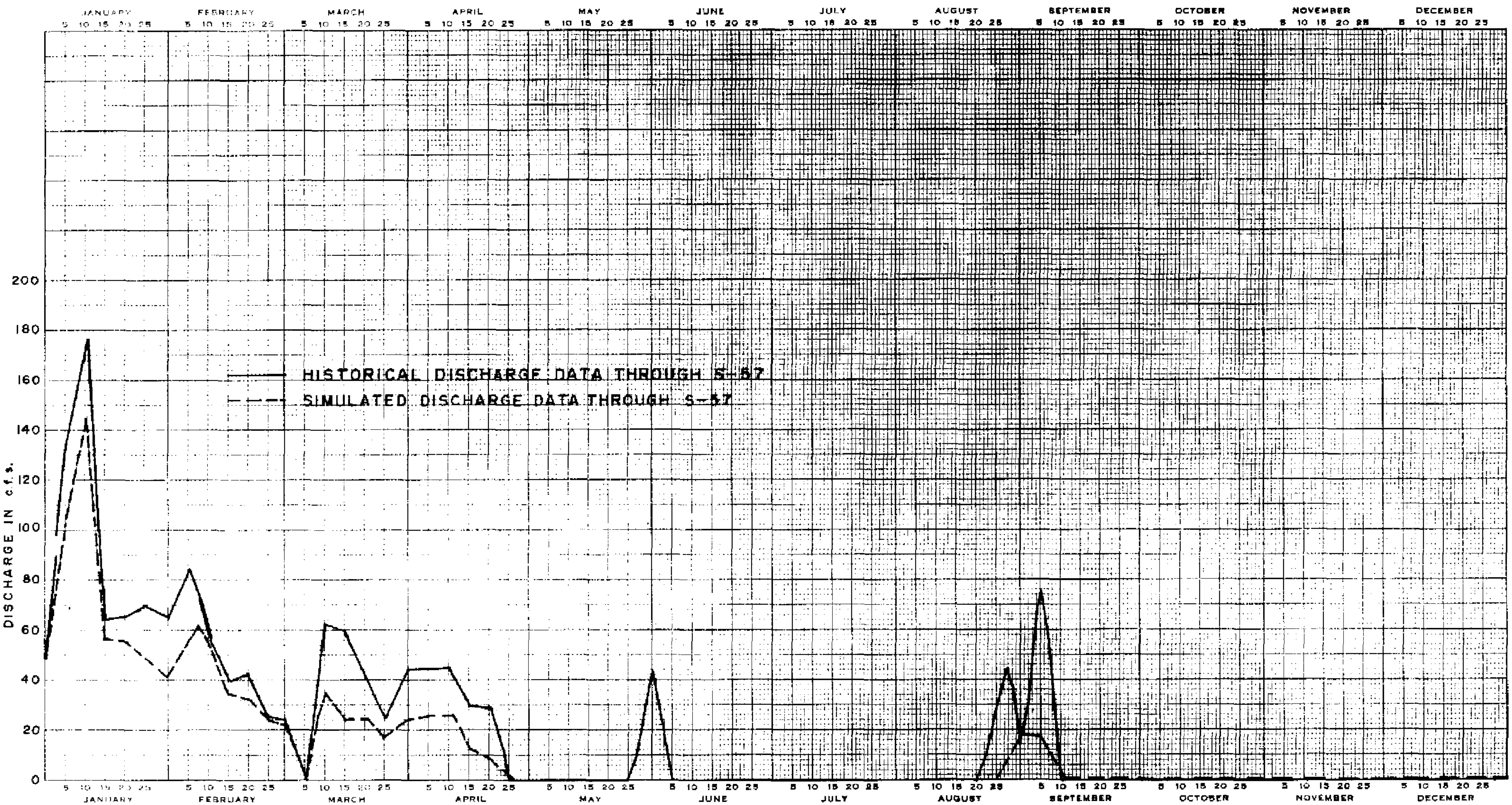
5. operational data such as gate openings and the corresponding headwater, tailwater elevations at all the 14 control structures of the upper and lower Kissimmee basin on a 3 hour basis.

A BRIEF DISCUSSION ON THE PRIMARY OUTPUT

Considering the fact that we are for the first time trying to incorporate collectively huge sets of input data, various types of coefficients, numerous mathematical formulations, complexities in computer logic and iterative procedures in developing an operational framework for the hydraulic simulation of the entire Kissimmee basin, the results are indeed encouraging. The success of our methodology can be viewed in terms of the various comparisons of simulated stages and discharges with the corresponding recorded values. Because of the interdependence between and within the various component parts of the overall operational watershed model, it is to be noted that a particular set of simulated stages and discharges correspond to the specific set of the combinations of coefficients, mathematical formulations and input data. Even if one coefficient is changed arbitrarily or systematically, the output of simulated stages and discharges corresponding to that change can be significantly different from the previous set. In addition there are numerous key coefficients, rate constants and numerical multipliers that can be changed. As a result, it seems justified to first obtain an output based on realistic and well documented input data set and then to proceed for further tuning-up of the proper coefficients in the right direction after comparing the simulated values with the recorded values.

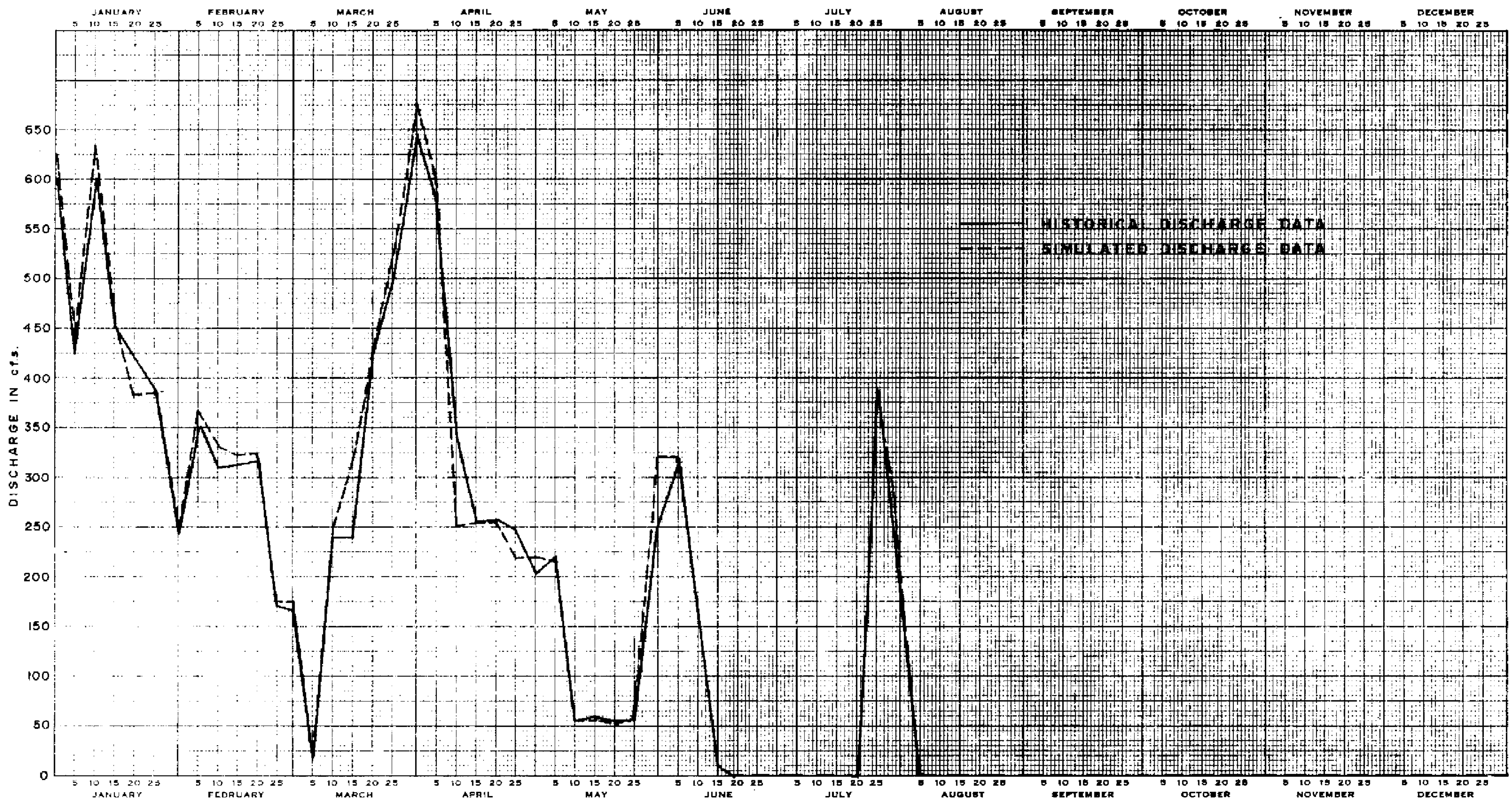
For example, the first-cut results of our routing methodology for the upper Kissimmee basin gave us comparative discharge graphs shown in figures 8-15 for a particular set of state conditions, basin parameters of sub-basin models coupled with another specific set of proportioning factors, tabular values and mathematical formulations of the routing model. Although the correlations depicted in figures 8-15 for discharges are excellent, the comparative graphs of simulated and recorded stages of some of the lakes of the upper Kissimmee show significant differences. To illustrate this point, typical results in the form of graphical comparison for lakes Tohopekaliga and East Tohopekaliga are depicted in figure 16. These preliminary comparisons indicate clearly

1. the capability of our overall framework of operational watershed model to combine the sub-basin model with the routing methodology and to generate the wanted simulated information on a first pass basis,
2. the relative importance of gate openings as against the head difference across the structure in the discharge rating formulations for the control structures,
3. the large simulated quantities of water coming into the upper Kissimmee system making the stages of lakes Cypress, Hatchineha, Kissimmee and Gentry to increase abnormally. Such a discrepancy of excessive water in the system is further investigated by examining the sub-basin model output and associated parameters. The overall framework of this model is set up in a manner in which it is possible to detect such abnormal behavior of the system and
4. the need of systematic and careful approach of performing parametric sensitivity analysis to improve and refine the output.



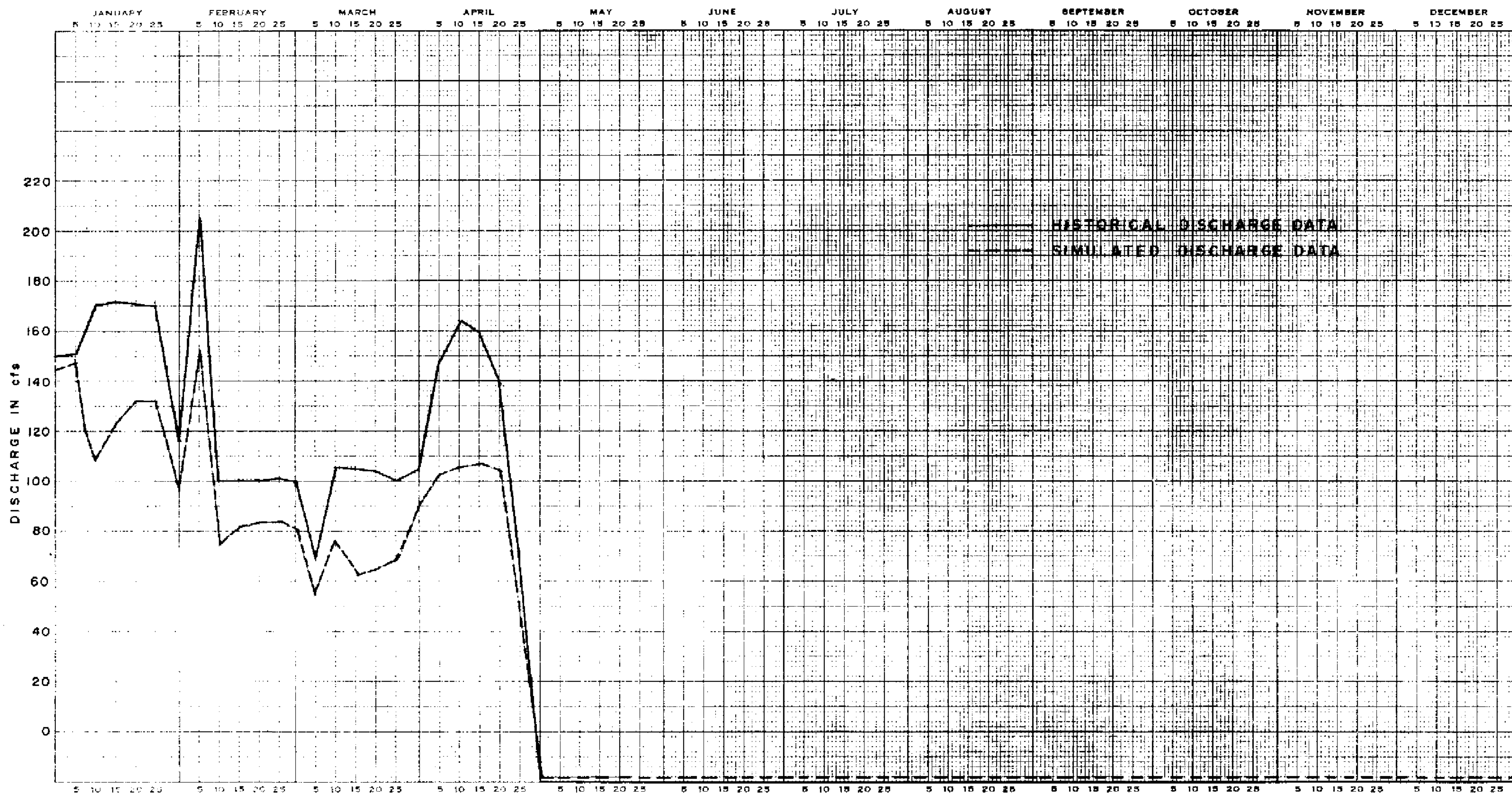
COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S-57 FOR THE FULL YEAR OF 1970

FIGURE 8



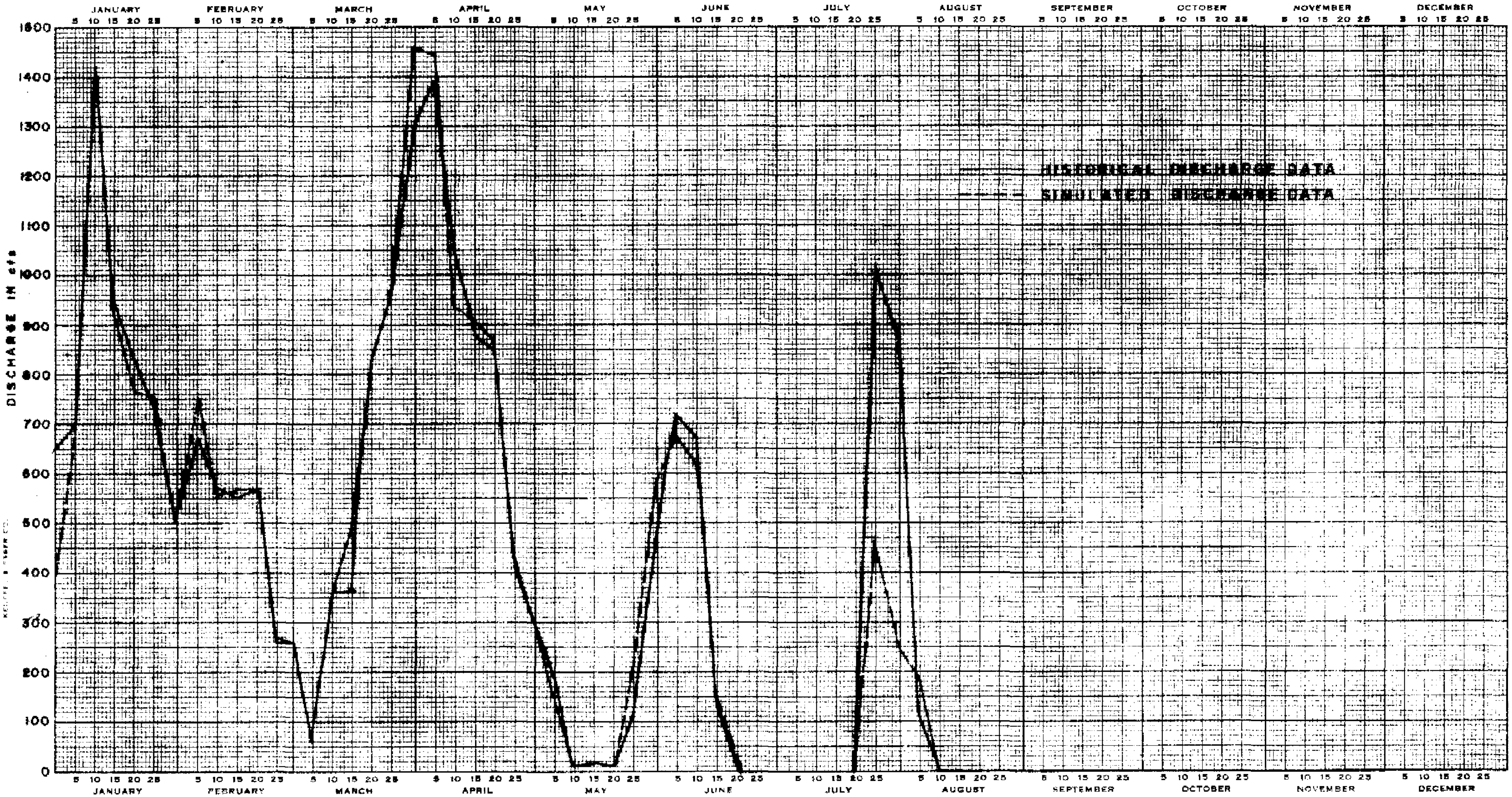
COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S-59 FOR THE FULL YEAR OF 1970

FIGURE 9



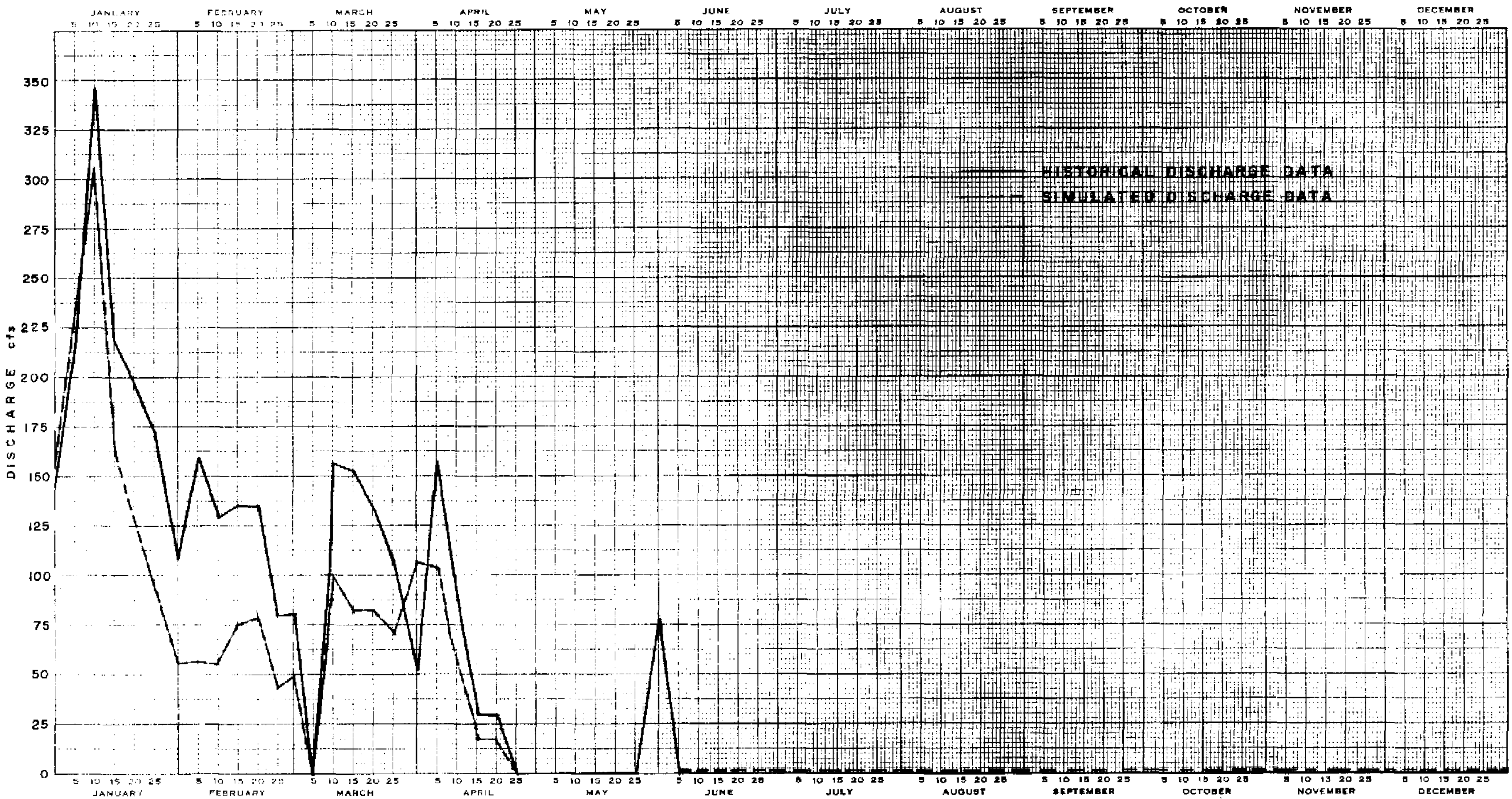
COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S-60 FOR THE FULL YEAR OF 1970

FIGURE 10



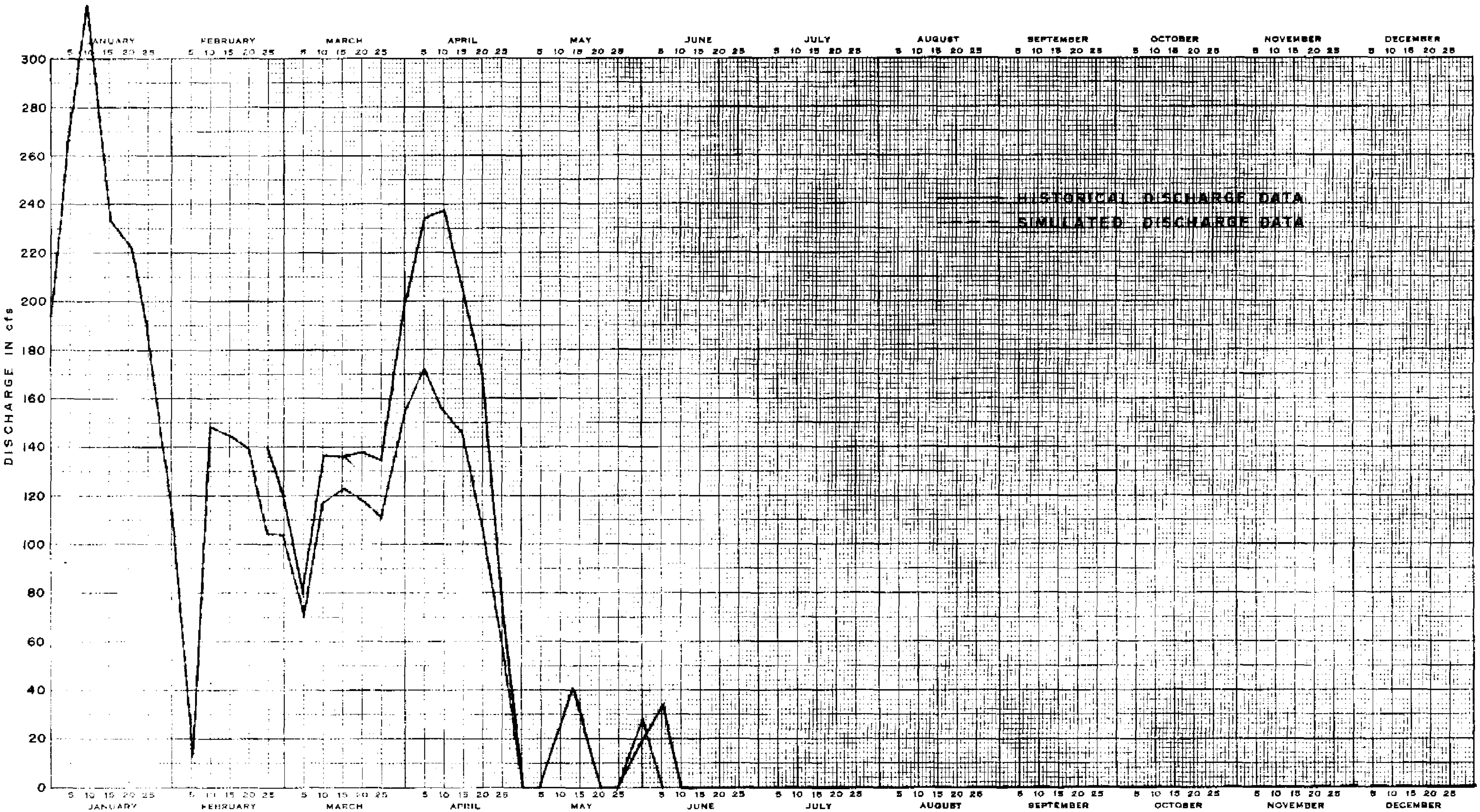
COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S-61 FOR THE FULL YEAR OF 1970

FIGURE 11



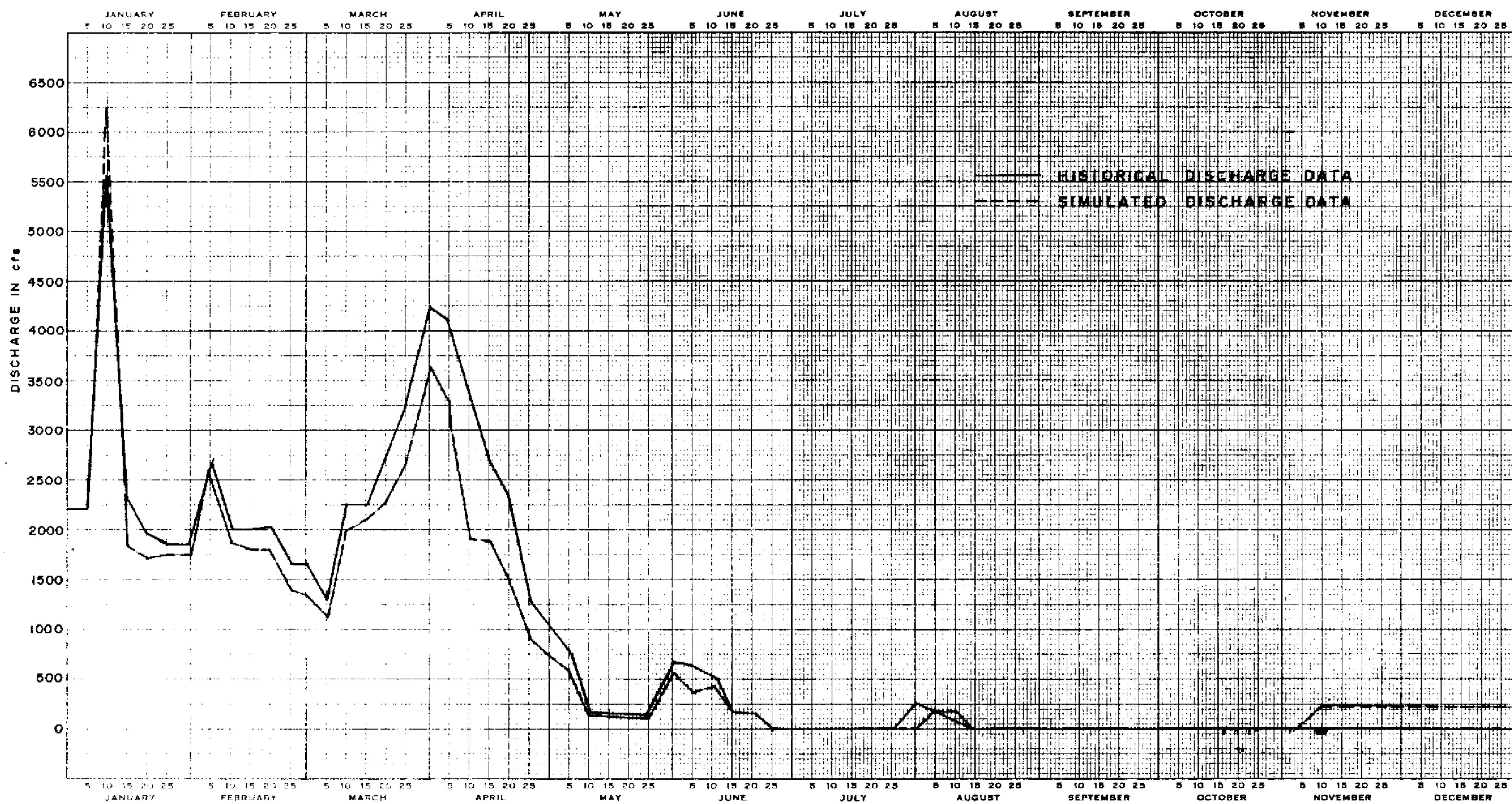
COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S—62 FOR THE FULL YEAR OF 1970

FIGURE 12



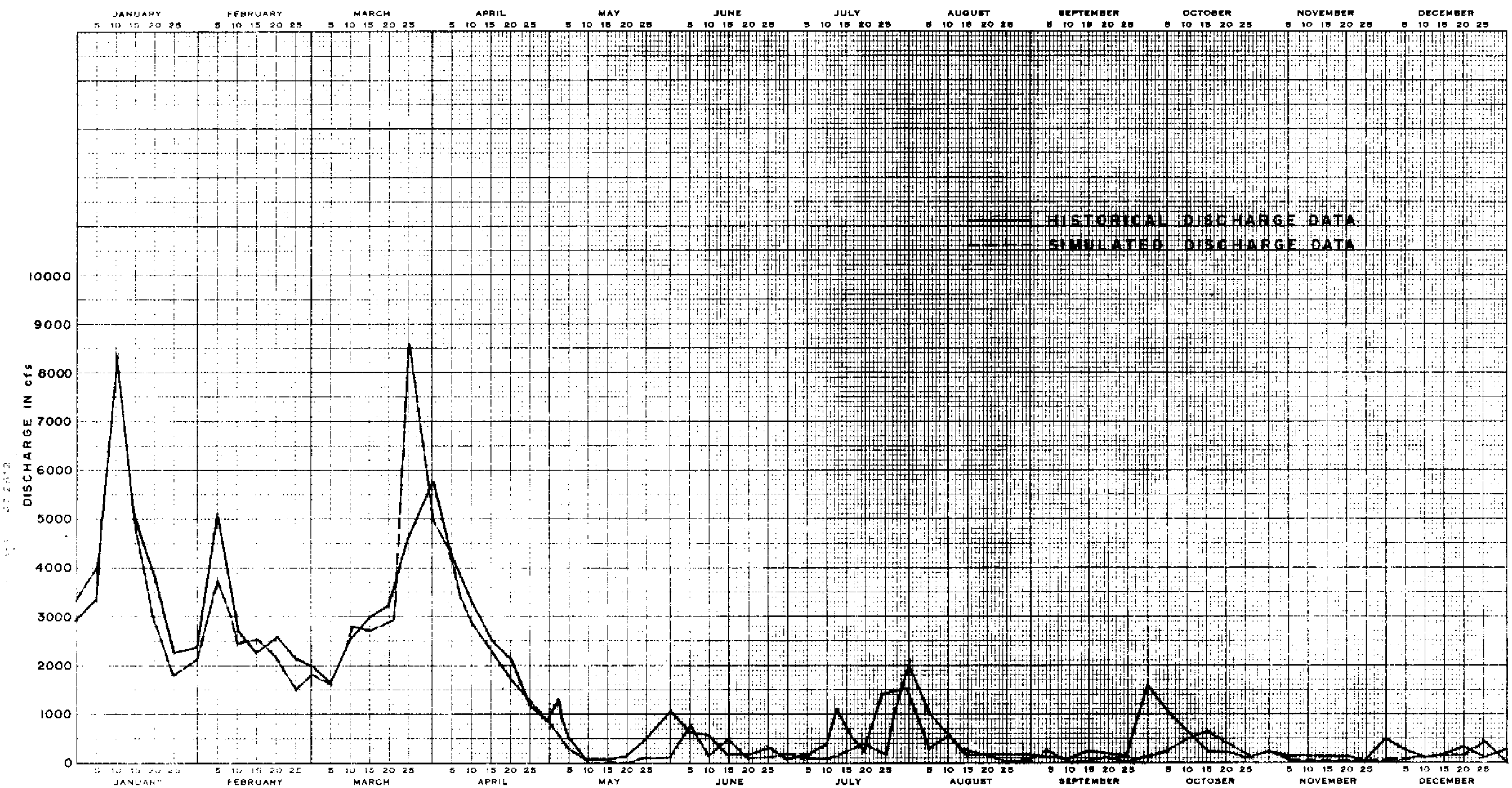
COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S-63 FOR THE FULL YEAR OF 1970

FIGURE 13



COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S-65 FOR THE FULL YEAR OF 1970

FIGURE 14



COMPARISON OF SIMULATED AND RECORDED DISCHARGES THROUGH S-65E FOR THE FULL YEAR OF 1970

FIGURE 15

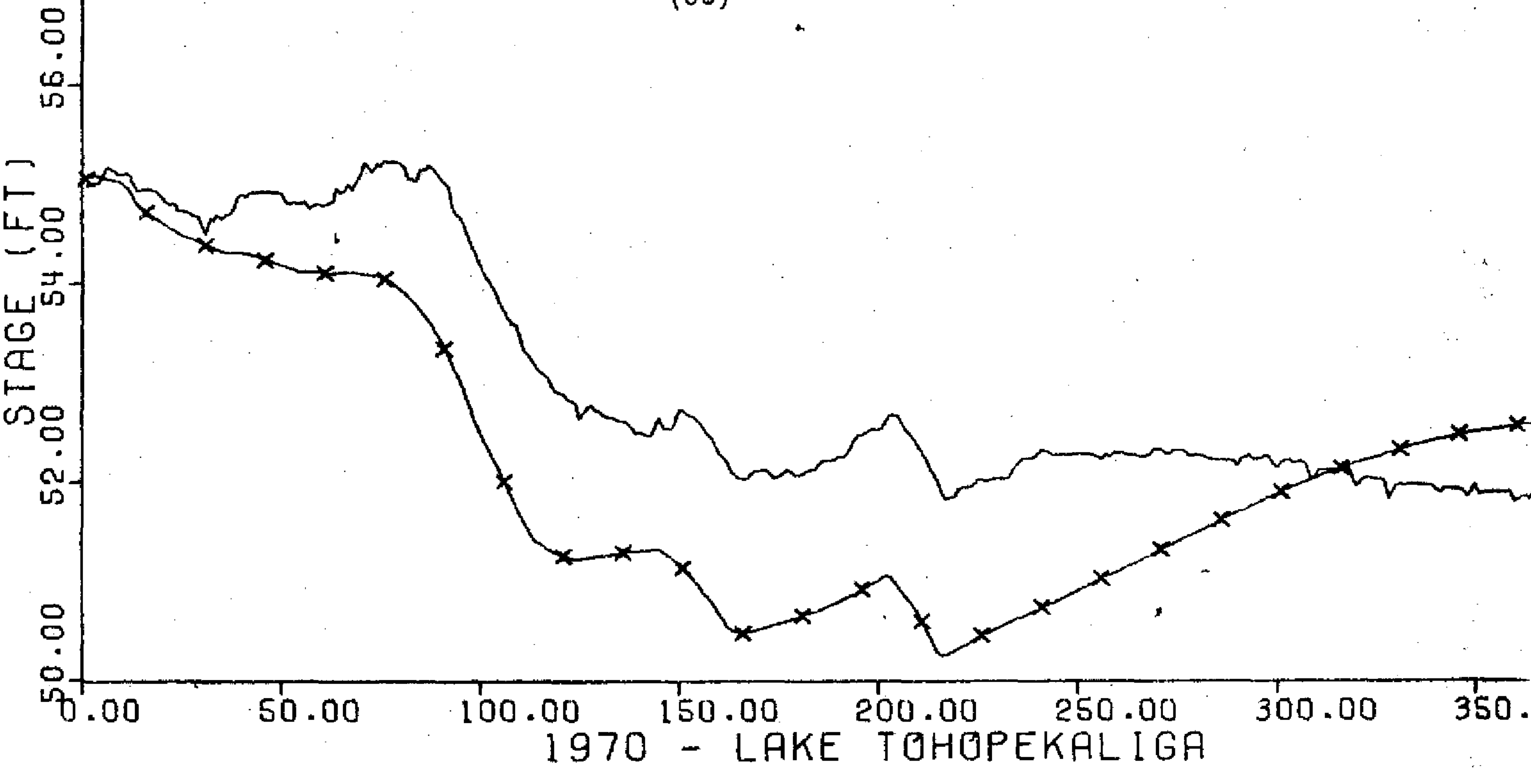
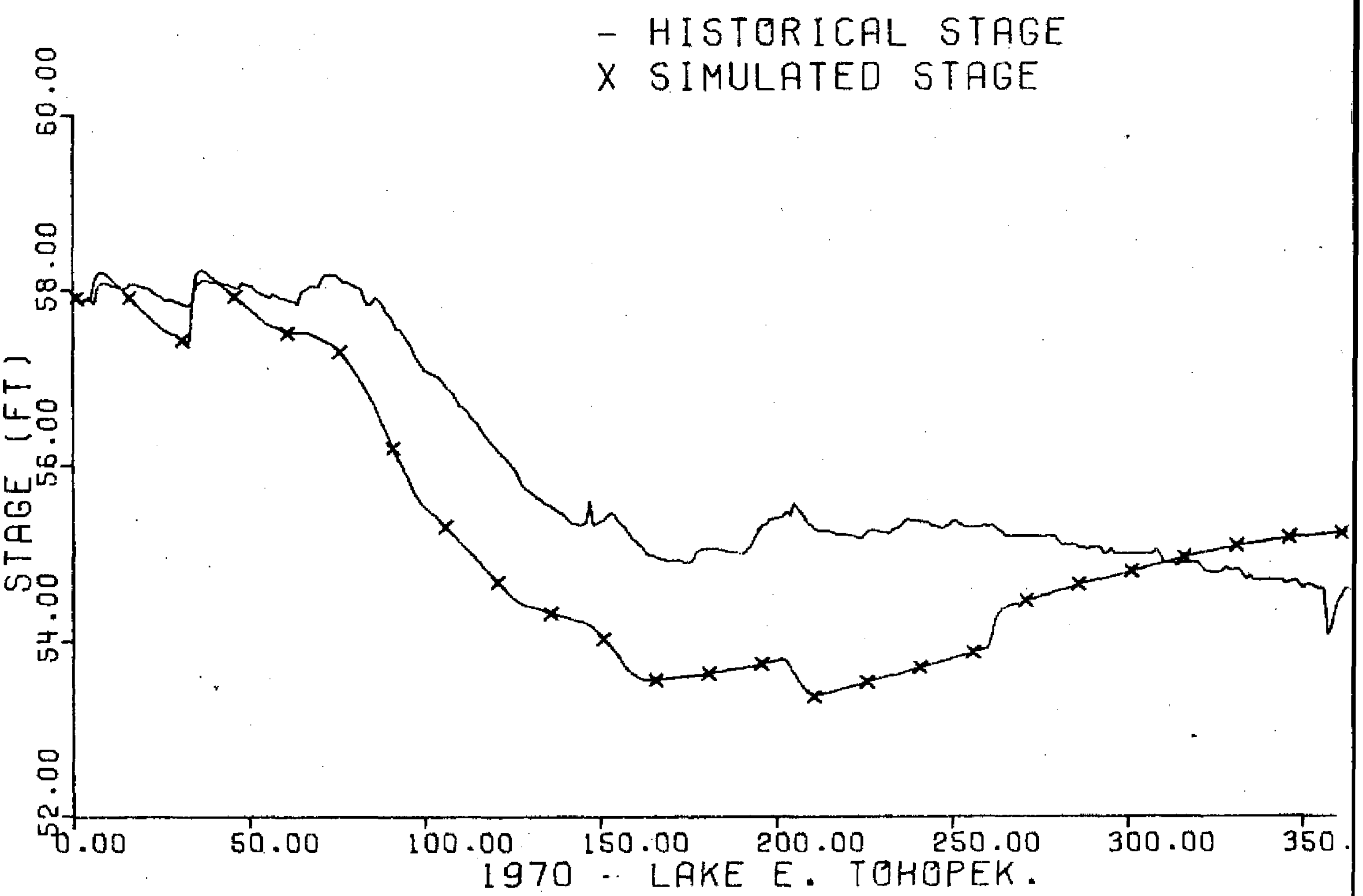


Figure 16. Comparison of simulated and recorded stages for Lakes Tohopekaliga and East Tohopekaliga for the year 1970.



CONCLUSIONS

1. After refining, modifying, formulating and designing various individual pieces of the operational watershed model, (also called water quantity model) a framework is developed for the Kissimmee basin to determine the hydrologic and hydraulic performance of its water systems under a given rainfall distribution. Due to the tedious and time-consuming task of gathering operational data on a 3 hour basis, the working of our quantity model (a combination of sub-basin model and routing model) is demonstrated for only one year of 1970, although such methodology can be extended to any number of years.
2. With a conventional framework of assumptions and simplifications the developed computer program (which takes about 5 hours per year of simulation) is designed to perform the operations:
    a. generating hydrologic parameters (such as sub-surface flow, surface flow, evaporation losses, deep seepage loss, available soil storage, storage in depression and finally streamflows) on a 3 hour basis for 19 planning units using rainfall, stage conditions and basin parameters as input data for 1970.
    b. routing these streamflows of the 19 planning units through the controlled systems of lake, channel and operating structures.
    c. simulating 3 hour lake stages, headwater and tailwater elevations of structures and discharges through the structures of the upper and lower Kissimmee using 3 hour gate operations data, initial conditions of stages, backwater formulations and the set of

proportioning factors as input data.

d. comparing the simulated values with recorded values in terms of plotted graphs and tables.

e. tuning-up the model to every extent possible by changing the various parameters of the sub-basin model and the routing model.

3. While developing the main framework of the quantity model, the following secondary tasks are also completed:

   a. processing the available cross-sectional data of all the pertinent sections of the upper and lower Kissimmee and putting them in a usable computer form to be able to use them directly in the existing FCD backwater program.

   b. developing the interrelationships between discharges, upstream and downstream stages for all the pertinent channel sections of the upper and lower Kissimmee basin and formulating the stage-storage characteristics of the upper Kissimmee lakes and five pools of the lower Kissimmee.

   c. generating 3 hour gate opening data for 14 control structures of the Kissimmee for the calendar year of 1970 and three hour tailwater and headwater stages at nine check points of S-58, S-57, S-62, S-59, S-61, S-63, S-60, S-65 and S-65E.

   It is expected that such a data base can be used in on-going and future studies on the Kissimmee River basin.

FURTHER AREAS OF INVESTIGATIONS

With adequate experience with the relative importance of the various pieces of the operational watershed model, it seems necessary to refine and pursue the following areas by further investigation:

1. Although the formulations used in this methodology are justified from statistical and hydraulic standpoints, another set of more accurate relationships can be developed by generating the backwater data for a further narrower practical range of stages and discharges around the operating conditions. It is possible to refine the existing mathematical formulation in these directions.
2. Due to the tedious and laborious task of collecting 3 hour gate operations data for all the control structures of the Kissimmee basin, our methodology is demonstrated for only one year, 1970. It is logical to extend this useful methodology (with necessary modifications in its pieces) for ten years (1960 to 1970). After such an extension, it is quite possible to simulate the hydrologic interactions of the Kissimmee basin with more valid hydrologic information for the various factors.
3. Although sensitivity analysis is performed on the various basin parameters of the sub-basin model, the procedures to determine the values of these basin parameters are taken for granted. With a view to further calibrate our model for more accuracy, it may be necessary to reexamine these procedures for more systematic parametric analysis.
4. The most useful extension of this effort is to be able to provide operational information regarding the essential gate openings at various control structures to keep the required water levels and flows in various sections of the Kissimmee water system.

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NOTATIONS USED IN THIS REPORT

ΔS	change in storage,
WSE	water surface elevation,
S	storage,
SO	bottom bed slope,
SE	slope of the energy line,
n	Manning's coefficient,
V	velocity
HR	hydraulic radius
Q	discharge
A	cross sectional area
Y	depth
GO	gate opening
EH	headwater elevation - tailwater elevation = head across the structure
α	velocity head coefficient
T	top width of the channel
g	gravitation acceleration
a, b, p, r s	constants
DX	distance between reaches i+1 and i
E069	the existing FCD computer program for the multivariate analysis
E070	the existing FCD computer program for the channel sectional analysis
E081	the existing FCD backwater program